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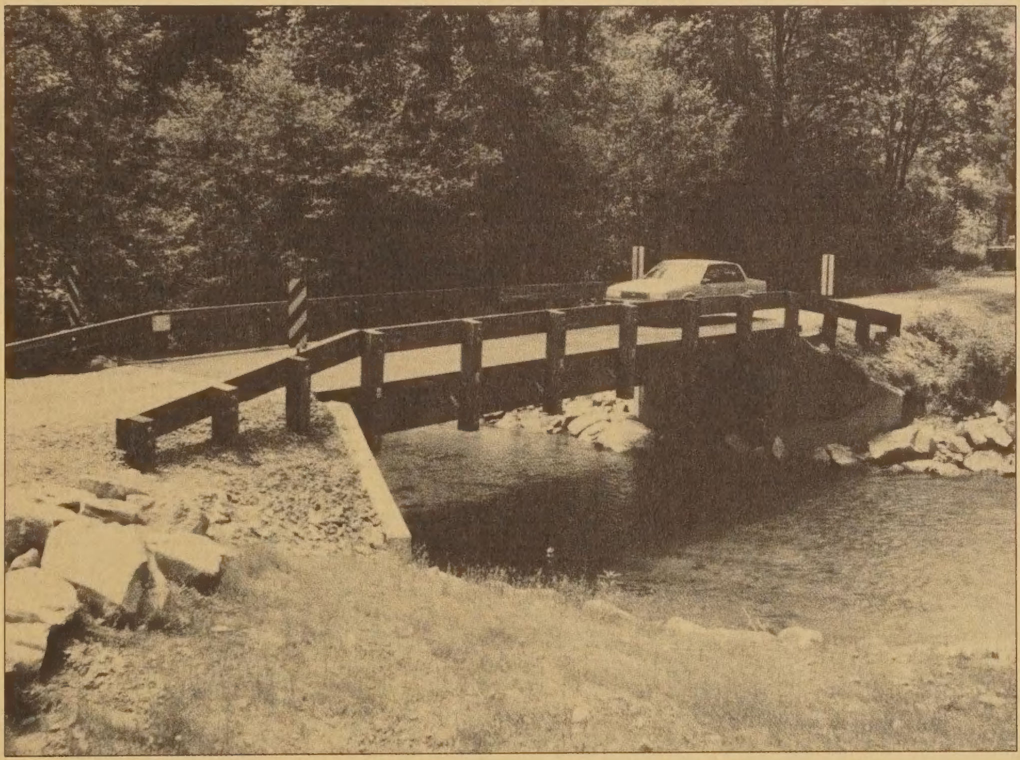
Materials for and Design of Hardwood Glulam Bridges



PREFACE

This publication is part of a technology transfer effort on timber bridge technology conducted by the USDA Forest Service, Timber Bridge Information Resource Center in cooperation with Dr. Harvey B. Manbeck and Mr. Keith Shaffer, Agricultural and Biological Engineering Department, The Pennsylvania State University. This publication was developed from a presentation at the "National Hardwood Timber Bridge Conference, 1992" held at The Pennsylvania State University.

This publication presents design requirements, calculations, and details for 30- to 40-feet simply supported hardwood glulam girder bridges with transverse glulam decks. The new hardwood glulam bridge standards being prepared for PennDOT are discussed. The design procedures are explained and are followed by design examples.

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Cover photo: *Bear Creek Run Bridge, Elk County, on the Allegheny National Forest. Construction of the Bear Creek Bridge represents the first application of glue-laminated (glulam) red maple hardwood for bridge deck and railings. Photo courtesy of Allegheny National Forest.*

Timber Bridge Information Resource Center

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National Hardwood Timber Bridge Conference 1992

Materials for and Design of Hardwood Glulam Bridges

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MATERIALS FOR AND DESIGN OF HARDWOOD GLULAM BRIDGES *

The goals of this manual are to:

1. Identify the characteristics and structural properties of hardwood glulam.
2. Present procedures for designing a typical hardwood glulam superstructure.
3. Introduce the PennDOT BLC-560 Series, "Standards for Hardwood Glulam Timber Bridge Design."
4. Summarize the design and performance of a hardwood glulam demonstration bridge.

Section 1: Hardwood Glulam — Its Characteristics and Properties

1.1 General

1.1.1 Definition of Glulam Lumber

Glulam lumber refers to structural members which are manufactured from 3/4-inch to 2-inch thick laminations of structural grade lumber (Figure 1.1). The laminations are cut to width and manufactured into lengths equal to the structural member length by end jointing shorter lengths of lumber. One method of end jointing is finger jointing (Figure 1.2). Finger joints are typically glued using a melamine adhesive. The glulam member is manufactured by gluing the faces with a water-resistant adhesive (typically a resorcinol-formaldehyde or a phenol-resorcinol-formaldehyde resin).

1.1.2 Advantages of Glulam

Glulam members have numerous advantages including:

- A variety of cross sectional dimensions and lengths can be manufactured using common size pieces of structurally graded lumber.
- Members can be fabricated using lower grade materials in the less stressed portions (e.g., the core of flexural cross sections) of a structural member (Figure 1.3).
- Allowable stresses of glulam are typically higher than allowable stresses of individual pieces of solid-sawn lumber of the same size and grade because strength reducing characteristics (e.g., knots) are dispersed throughout the cross sections rather than being concentrated at a single cross section (Figure 1.4).

*** A video tape based on the information in this publication is available from Agricultural Information Services, 119 Agricultural Administration Building, Penn State University, University Park, PA 16802 (Phone: 814-865-6309).**

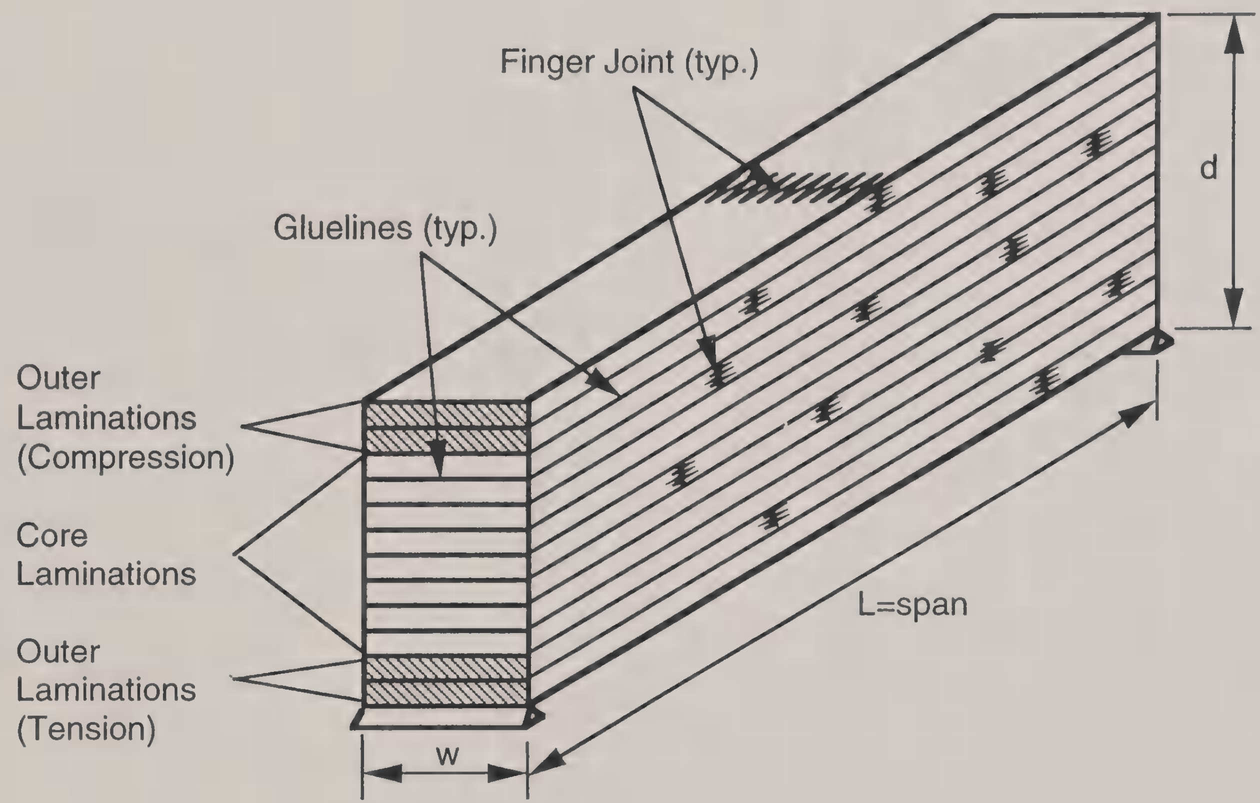


Figure 1.1 Glulam timber definition sketch.

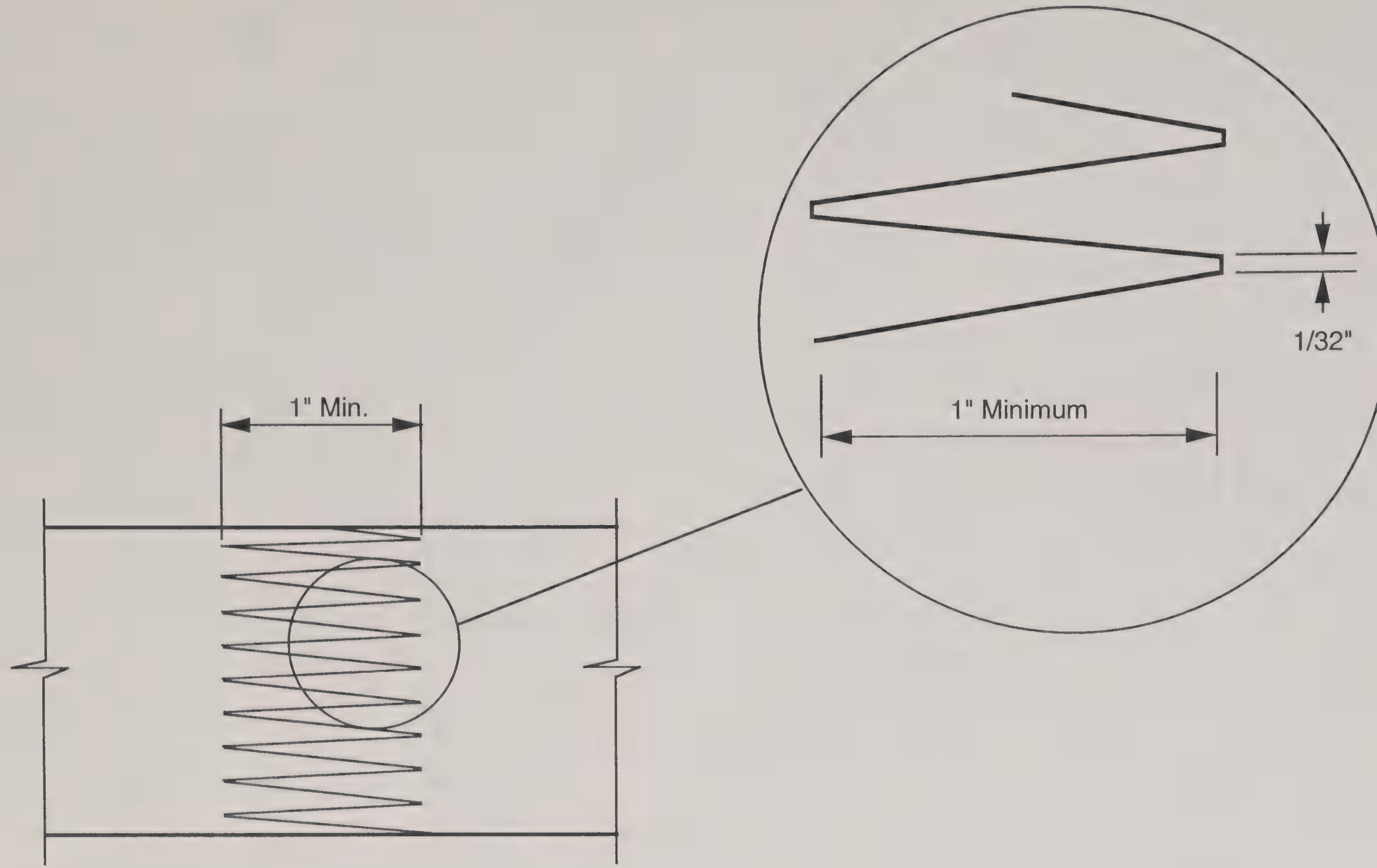
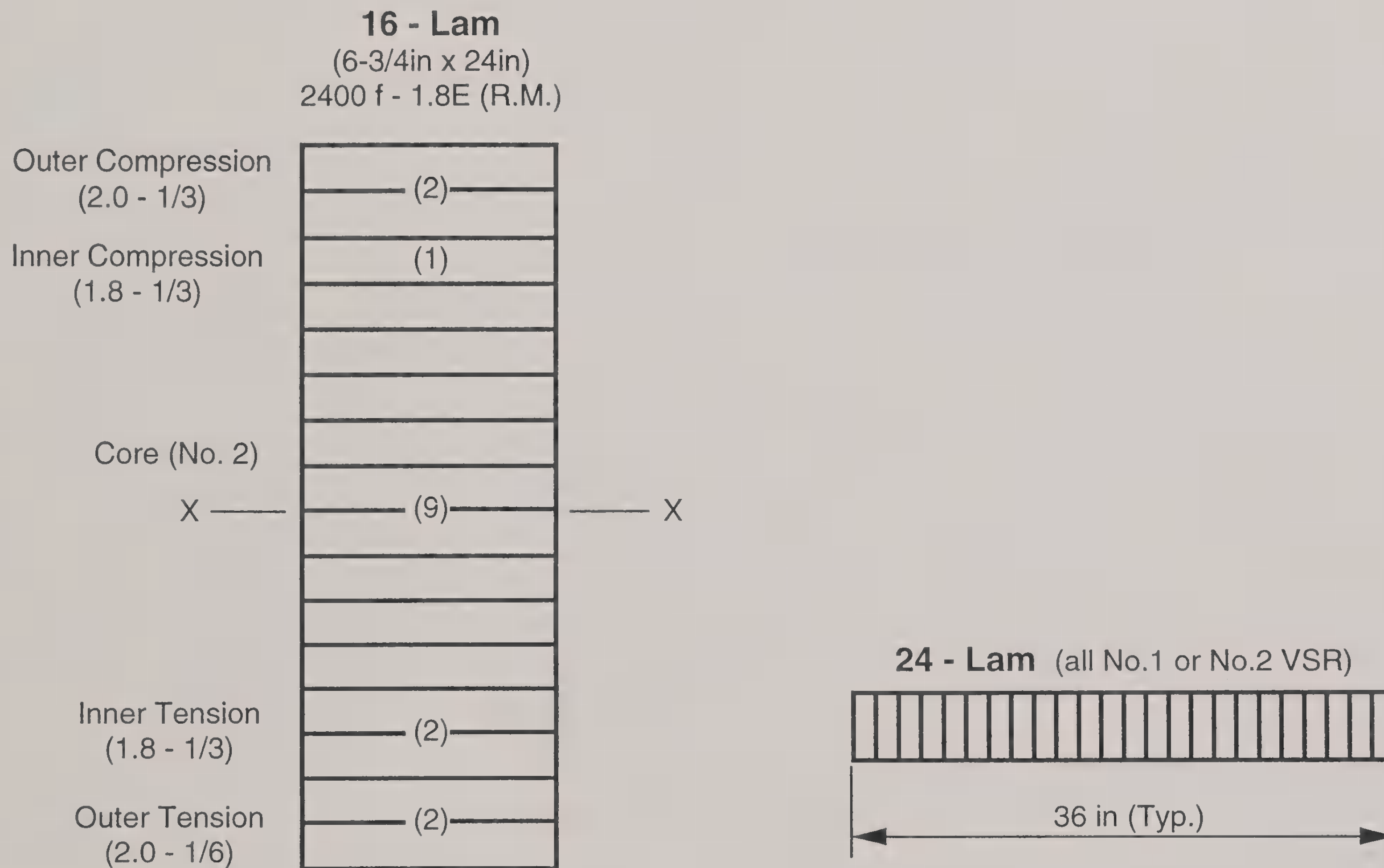


Figure 1.2 Typical finger construction joint detail.



(a) Stringer element
(Loaded perpendicular to faces)

(b) Deck element
(Loaded parallel to faces)

Figure 1.3 Typical glulam cross-sections.

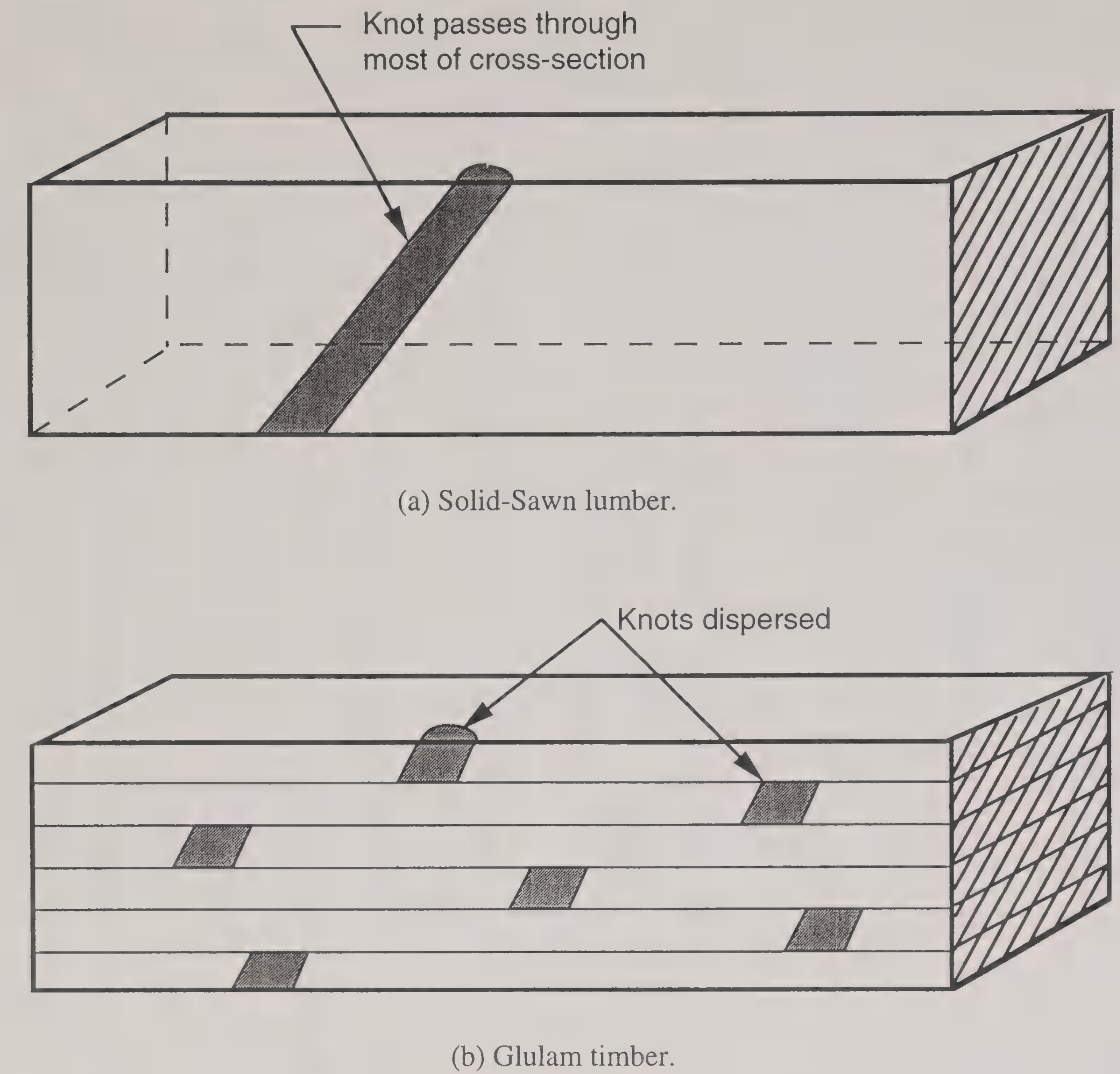


Figure 1.4 Dispersion of strength reducing characteristics (e.g. knots, slope of grain).

1.1.3 Quality Control

Glulam quality depends upon use of the proper grades of lumber in the cross section layup and satisfactory gluing of the lamination end joints and lamination faces. Quality is best assured by manufacture of the members at a properly certified laminating plant. Typical certification agencies include AITC (American Institute of Timber Construction) and APA (American Plywood Association). The certified manufacturing plant has a standard quality control program. The certifying agency periodically inspects the plant to assure that quality standards are met.

The certified manufacturing plant manufactures and fabricates the glulam member in accordance with the following standards.

- ANSI/AITC A190.1-1983. 1983. American National Standard for Wood Products--Structural Glued Laminated Timber. AITC. Vancouver, WA.
- AITC 200-83. 1983. Inspection Manual. AITC. Vancouver, WA.
- AITC 117-88. 1988. Manufacturing: Standard Specifications for Structural Glued Laminated Timber of Softwood Species. AITC. Vancouver, WA.
- AITC 117-87. 1987. Design: Laminating Specifications. AITC. Vancouver, WA.
- AITC 119-85. 1985. Standard Specifications for Hardwood Glued Laminated Timber. AITC. Vancouver, WA.

1.1.4 Allowable Design Values (ADV) of Glulam Timbers

Allowable design values (ADV) for strength and stiffness of softwood glulam timbers are published in AITC 117-87 -- "Design: Laminating Specifications." Included are specific values for several softwood species and for a wide variety of lamination combinations. ADV's of hardwood glulam members and numerous lamination layup combinations are published in AITC 119-85 -- "Standard Specifications for Hardwood Glued Laminated Timber." The ADV's for softwood glulam members are summarized in Tables 5A and 5B, pages 42 to 50, of the *NDS® Supplement to the 1991 Edition of the National Design Specification for Wood Construction: Design Values for Wood Construction*. The ADV's for hardwood glulam members are summarized in Table 5C, page 51, of the NDS Supplement (1991 Ed.). Current hardwood glulam specifications are published for layup combinations of uniform grade laminations. Proposed changes to AITC 119-87 as a result of research discussed in the manual will expand the number, flexibility, and efficiency of hardwood layup combinations.

1.2 Interpretation of Glulam ADV's

1.2.1 ADV's for Flexure from Testing

The general procedures for load-testing glulam beams and calculating flexural ADV's from the data are outlined in this section. The development herein is conceptual in nature and does not delve into the details of the methods. Detailed standard procedures are included in:

- ASTM D198-84. 1984. Standard Method for Static Tests of Timbers in Structural Sizes. ASTM. Philadelphia, PA.

- ASTM D2915-84. 1984. Standard Method for Evaluating Allowable Properties for Grades of Structural Lumber. ASTM. Philadelphia, PA.

The generalized procedural steps are:

Step 1. A sample of test glulam beams of the specified layup combination is manufactured in accordance with ANSI/AITC 190.1 and AITC 119 or AITC 117. Sample size requirements depend upon the variability of test data. Sample sizes of 15 or more are often suggested for glulam. Care is taken to proportion the beam properly for testing.

Step 2. Each test beam is loaded to fracture using a two-point loading scheme (Figure 1.5a). The modulus of rupture (MOR) of the beam is calculated with equation (1).

$$MOR_i = M_u/S = P_u\, a/S \qquad (1)$$

M_u = ultimate moment (lb.-in.)

P_u = ultimate load (lb.)

S = section modulus (I/c) (in.^{3})

The modulus of elasticity (MOE) of the specimen is calculated using the equation for the midspan deflection of an elastic beam. A data point, such as (P_1, Δ_1) in Figure 1.5b is used to calculate MOE for the beam. The MOR's and MOE's for wood products are typically distributed normally or log-normally (Figure 1.6).

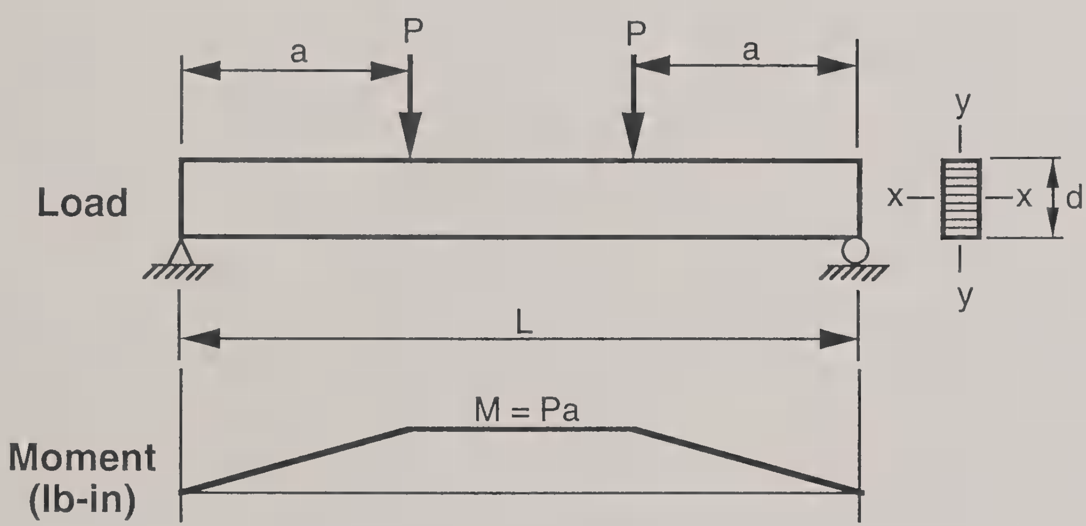


Figure 1.5a Test load and moment diagrams.

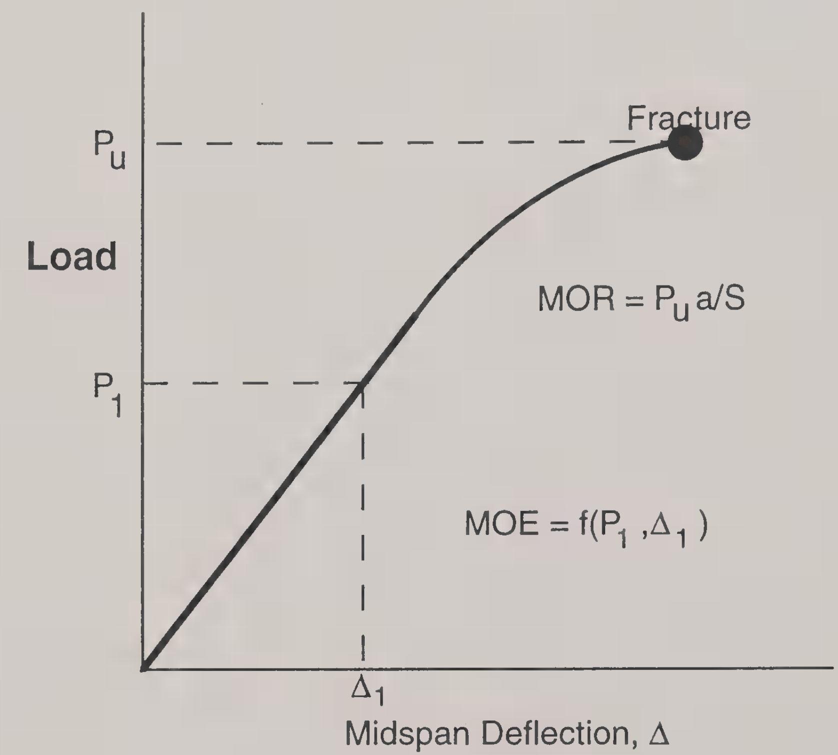


Figure 1.5b Schematic of load-midspan deflection curve.

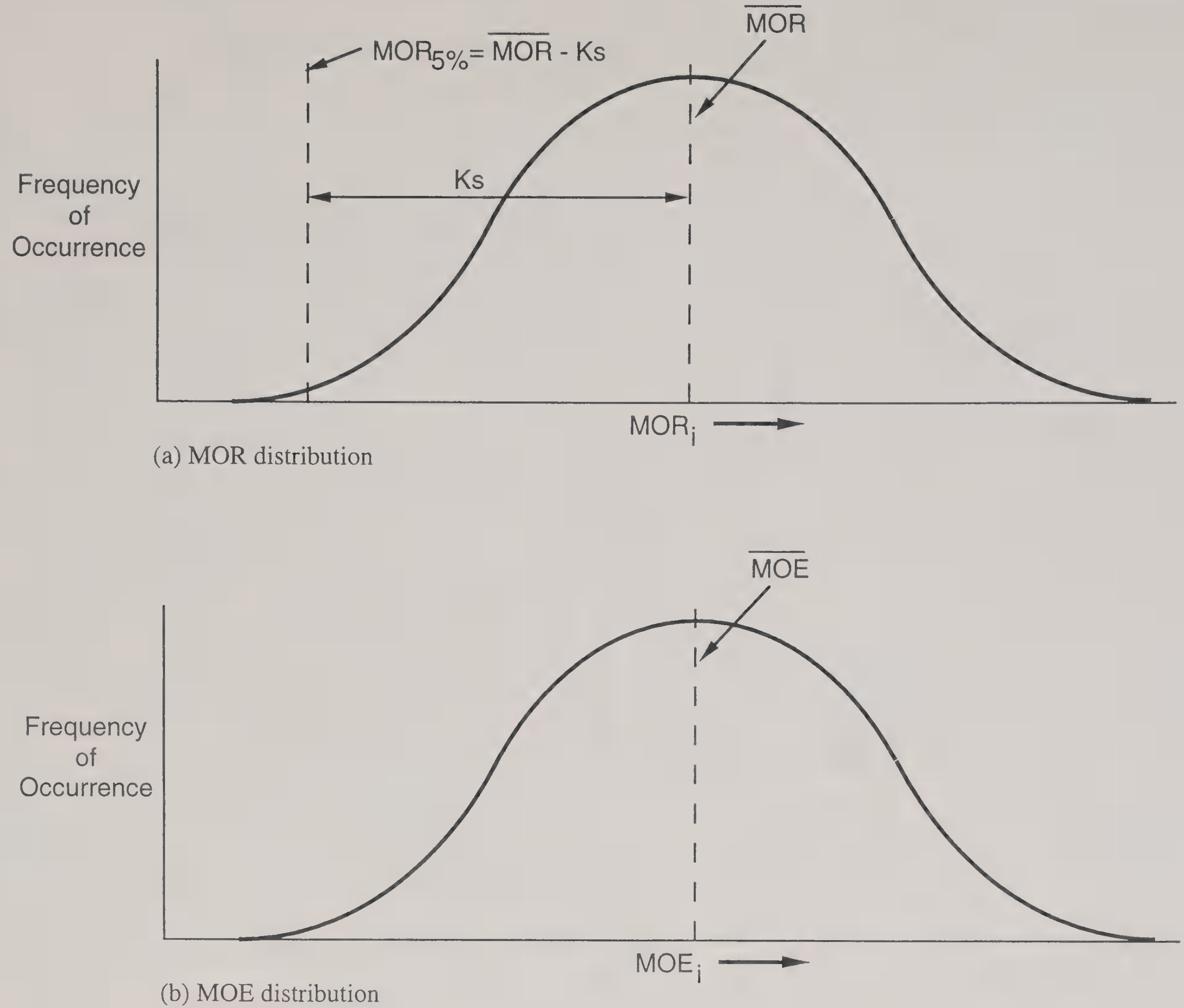


Figure 1.6 Distribution of test specimen MOR- and MOE- values.

Step 3. The mean and standard deviation of the MOR-distribution are calculated using equations (2) and (3); the mean of the MOE distribution is calculated using equation (4).

$$\overline{MOR} = \sum_i (MOR_i)/n \qquad (2)$$

where $\overline{MOR}$ = mean MOR (PSI)

n = number of test beams

$$s = \sqrt{\frac{\Sigma\left(MOR_i - \overline{MOR}\right)^2}{n-1}} \qquad (3)$$

where s = standard deviation (psi)

$$\overline{MOE} = \Sigma (MOE)_i / n \qquad (4)$$

where $\overline{MOE}$ = mean MOE (psi)

Step 4. The lower 5th percentile MOR-value at a specified tolerance limit is computed using equation (5). A tolerance limit of 75% is usually used for glulam.

$$MOR_{5\%} = \overline{MOR} - (K)(s) \qquad (5)$$

where $MOR_{5\%}$ = lower 5th Percentile MOR (Fig. 6a) (psi)

K = statistical parameter dependent upon sample size and probability level (Table 1.1).

Table 1.1. K-values.

Sample Size	75% Tolerance Limit	95% Tolerance Limit
15	1.991	2.566
30	1.869	2.220
45	1.821	2.092
50	1.811	2.065

1. From ASTM-D2915 (Table 3).

Step 5. Compute the Allowable Flexural Strength, F_{bx}, using equation (6).

$$F_{bx} = \frac{1}{2.1}(MOR_{5\%}) \qquad (6)$$

The value, 2.1, is a reduction factor which relates ultimate strength test data to allowable properties for design.

Step 6. The allowable stiffness for the layup is the average value equation (7).

$$E_x = \overline{MOE} \qquad (7)$$

1.2.2 Other ADV's from Test Data

The procedures for determining other ADV's (F_{by}, E_y, etc.) from appropriate test data are similar. The reduction factor in Equation 6, however, varies with the property being calculated (Table 1.2).

Table 1.2. Reduction Factors.

Property (ADV)	Reduction Factor[1]
MOE--modulus of elasticity	1.0
F_b - flexural strength	2.1
F_t - tensile strength	2.1
F_c - compressive strength parallel to grain	1.9
F_v - shear strength	4.1
$F_{c\perp}$- compressive strength perpendicular to grain	1.5

[1]ASTM 2915 (1984)

1.2.3 ADV's from Calculation Procedures

Allowable design values for glulam timbers can also be calculated using ASTM D3737, "Standard Method for Establishing Stresses for Structural Glued-Laminated Timber (glulam)." The calculation procedures require that the grade and species of all the laminations in the layup be specified and known. ASTM D3737 and AITC calculation procedures allow prediction of ADV's for F_b (flexural strength), F_c (compressive strength parallel to the grain), F_{cp} (compressive strength perpendicular to the grain), F_v (shear strength), F_t (tensile strength), and E (modulus of elasticity).

The ASTM D3737 calculation procedures are well established and verified for softwood glulam timbers and are used to predict the ADV's for the softwood lamination layup combinations listed in AITC 117 and Tables 5A and 5B of the NDS Supplement. The ASTM 3737 procedures have not, until recently, been used to generate published ADV's for practical hardwood lamination layup combinations.

1.3 ADV's for Hardwood Glulam

1.3.1 Current AITC-119 and NDS Specifications

The current AITC 119 identifies ADV's for 14 hardwood species groups and five lamination combination layups (Combinations A, B, C, D, and E). Combination layups A-E consist of single-grade laminations characterized by maximum allowable knot sizes and maximum allowable slopes of grain. Knot sizes and slopes of grain limitations are summarized in Table 1.3. Note that the slope of grain requirement is more stringent (1:16) in the outer 10% of the cross section (Figure 1.7).

Table 1.3. Knot Size and Slope of Grain Limitations for Hardwood Combinations A through E.

Hardwood Combination	Maximum[1] Knot Size	Maximum Core Slope of Grain	Maximum Outer 10% Slope of Grain
A	0.1 w	1:8	1:16
B	0.2 w	1:8	1:16
C	0.3 w	1:8	1:16
D	0.4 w	1:8	1:16
E	0.5 w	1:8	1:16

[1]w = face width of lamination

ADV's for hardwood are obtained using the data in Table 5C on page 51 of the NDS® Supplement and equation (8).

$$\text{ADV} = (\text{Factor A})\ (\text{Stress Module}) \qquad (8)$$

Factor A = tabulated factor for wood species in Part A of Table 5C of NDS Supplement
Stress Module = tabulated value in Part B of Table 5C of NDS® Supplement

For example, ADV's for Combination A red oak, including northern red oak (NRO), are:

$F_{bx} = F_{by} = (2.80)\ (800) = 2240$ psi
$F_t = (2.80)\ (500) = 1400$ psi
$F_c = (2.05)\ (970) = 1988$ psi
$E = (1.60)\ (1 \times 10^6) = 1.6 \times 10^6$ psi
$F_v = (1)\ (230) = 230$ psi
$F_{c\perp} = (1)\ (800) = 800$ psi

ADV's for Combinations A and B red oak and yellow poplar members are summarized in Table 1.4. Red maple, a soft maple, is not included in the current edition of AITC 119 nor in the 1991 NDS® supplement.

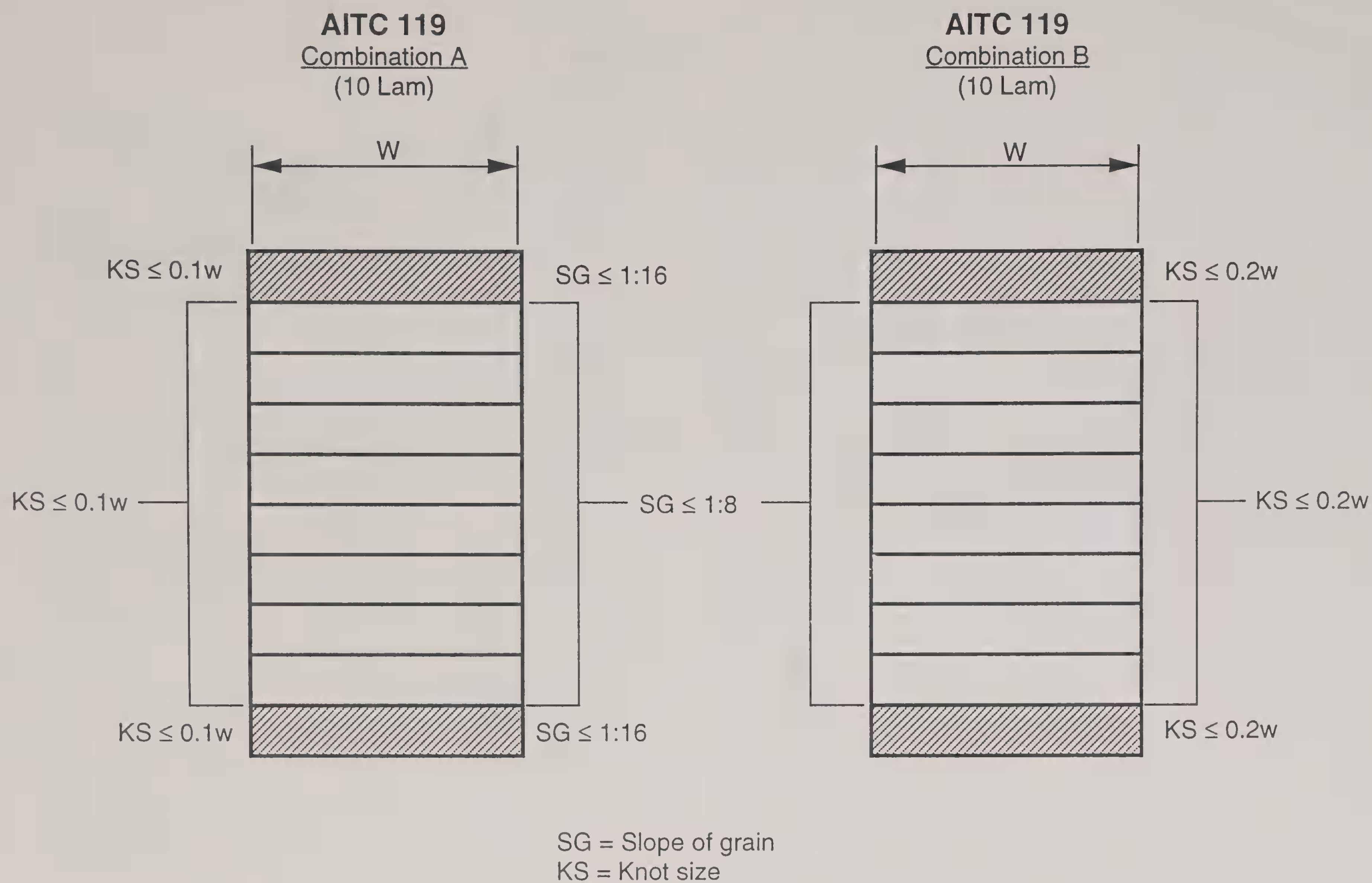


Figure 1.7 AITC 119 Combination A and Combination B hardwood layups.

Table 1.4. ADV's for Combination A and B Red Oak and Yellow Poplar Glulams (psi).

Property	Combination A Red Oak	Combination A Yellow Poplar	Combination B Red Oak	Combination B Yellow Poplar
F_{bx}	2240	1600	2156	1540
F_{by}	2240	1600	2156	1540
E_x	1.6×10^6	1.5×10^6	1.6×10^6	1.5×10^6
E_y	1.6×10^6	1.5×10^6	1.6×10^6	1.5×10^6
F_t*	1400	1000	1400	1000
F_c*	1988	1410	1880	1330
$F_{c\perp}$	800	410	800	410
F_v	230	150	230	150

* Note that there are special slope of grain requirements for F_t and F_c (See footnotes to Table 5C of NDS® Supplement).

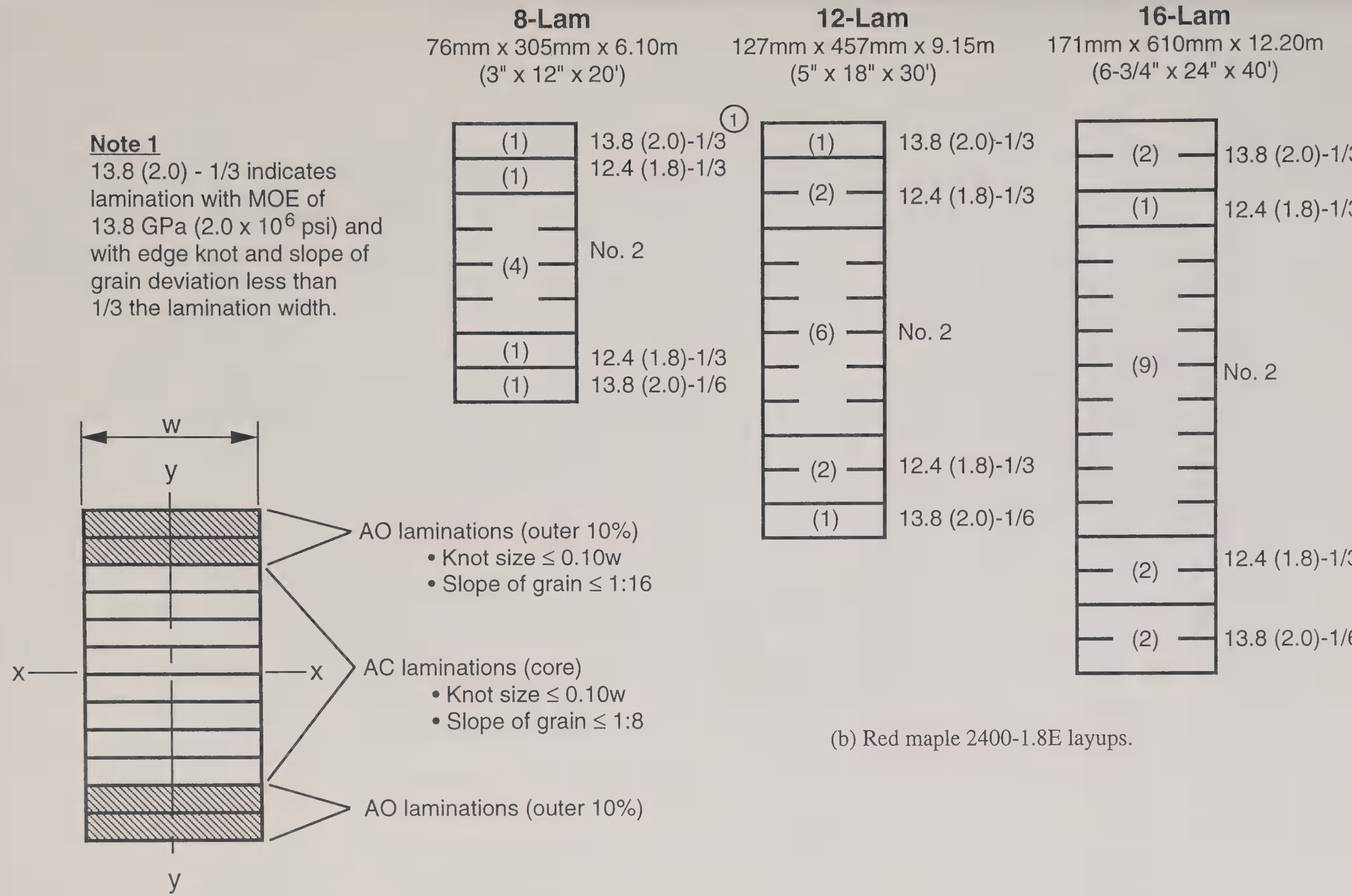
1.3.2 Recent Research on ADV's for Hardwood Glulam Members

1.3.2.1 Overview

Allowable design values were determined for Combination A northern red oak, red maple, and yellow poplar glulam beams loaded perpendicular to the plane of the laminations (Bending about the x-axis in Figure 1.8a) and parallel to the plane of the laminations (Bending about the y-axis in Figure 1.8a). Allowable flexural strengths (F_{bx}) and stiffnesses (E_x) were calculated from experimental load-deformation data for at least one lamination lay-up for each species.

Allowable flexural strengths (F_{by}) and stiffnesses (E_y) were also calculated from load-deformation data for one lamination lay-up of northern red oak and red maple. All other ADV's, including those for shear and bearing strength, were estimated and deduced from published values for the predominant grade of the lumber species used in the beam lamination lay-up.

After fabrication, the girders and deck panels used in hardwood timber bridges are treated with creosote. To obtain the treatment retention levels required by AWPA (1989), normal treatment pressures and temperatures had to be modified. Thus, it was necessary to determine if the post-fabrication treatment of hardwood glulam beams with creosote to AWPA retention levels adversely affected the strength or stiffness. An experiment was conducted to test the hypothesis that preservative treatment had no effect on the strength or stiffness of northern red oak, red maple, or yellow poplar glulam beams.



(b) Red maple 2400-1.8E layups.

(a) Northern Red Oak Combination A layup.

Figure 1.8 Combination A NRO and 2400 f-1.8E Red Maple Layups.

Research was also conducted to determine if: (1) The methods outlined in ASTM 3737 (ASTM, 1989) to predict the flexural strength and stiffness of softwood glulam beams are applicable to hardwood glulam beams; (2) It is technologically feasible to design and fabricate hardwood glulam beams with F_{bx} = 16.5 MPa (2400 psi) and E_x = 12.4GPa (1.8 x 10^6 psi); (3) The volume reduction effect for hardwood glulam beams is the same as that defined for softwood glulams in *the National Design Specification for Wood Construction* (NFPA, 1991). This phase of the study focused on red maple and yellow poplar.

Combination A lay-ups, as defined in the *National Design Specification for Wood Construction* (NFPA, 1991) and AITC-119 (AITC, 1986), were used for all the northern red oak studies (Figure 1.8a). Treatment effects for red maple and yellow poplar glulam beams were also evaluated using combination A lay-ups. ADV's were measured and predicted by ASTM D3737 (1989) for approximately 15 beams of each of the lay-ups shown in Figure 1.8b for red maple and yellow poplar.

1.3.2.2 Summary of Results

Comprehensive discussions of the methods and results of the treatment effect and ADV research are included in several research reports and articles (Manbeck, et al. 1993, Shaffer et al. 1991; Manbeck et al. 1994; Moody et al, 1993). The key results are summarized herein:

- Post-fabrication treatment with creosote to retention levels specified by AWPA (1989) did not adversely affect the flexural strength or the stiffness of northern red oak, red maple, and yellow poplar glulam beams.

- The measured dry-use flexural strength (F_{bx}) of combination A northern red oak glulam beams exceeded the values published in the NDS® (NFPA, 1991) for generic red oak. Measured allowable flexural strength for 40 beams was 23.6 MPa (3420 psi); published NFPA, 1991) values for red oak glulams is 15.4 MPa (2240 psi). An allowable value of 16.5 MPa (2400 psi) is recommended for design.

- The measured dry-use stiffness (E_x) of combination A northern red oak glulam beams exceeded NDS (NFPA, 1991) published values for generic red oak. The measured allowable stiffness was 13.1 GPa (1.9 x 10^6 psi); the published (NFPA, 1991) value for red oak is 11.0 GPa (1.6 x 10^6 psi). An allowable value of 12.4 GPa (1.8 x 10^6) psi is recommended for design.

- The allowable dry-use flexural strength (F_{bx}) of both red maple and yellow poplar lamination lay-ups shown in Figure 8b exceeded 16.5 MPa (2400 psi) and were satisfactorily predicted by the methods outlined in ASTM D3737 (1989).

- The allowable dry-use stiffness (E_x) of both red maple and yellow poplar lamination lay-ups shown in Figure 8b equaled 12.4 GPa (1.8 x 10^6 psi) and was satisfactorily predicted by the methods outlined in ASTM D3737 (1989).

- ASTM D3737 (1989) can be used to design red maple and yellow poplar glulam beam cross sections with specified strength and stiffness.

- The volume effect for red maple and yellow poplar glulam beams is similar to that for softwood glulam beams. That is, flexural strength (F_{bx}) declines as beam volume increases. Stiffness (E_x) is unaffected by volume. The volume reduction factor (C_v) is defined by equation (9) for both red maple and yellow poplar beams.

$$C_v = \left(\frac{b_o}{b}\right)^x \left(\frac{L_o}{L}\right)^y \left(\frac{d_o}{d}\right)^z \qquad (9)$$

where

b = cross section width, mm (in.)
d = cross section depth, mm (in.)
L = beam length, m (in.)
b_o = 130 mm (5.125 in.)
d_o = 305 mm (12 in.)
L_o = 6.38 m (21 ft.)
$x=y=z$ = 0.071 to 0.088 (red maple and yellow poplar, respectively)

1.3.3 Proposed Changes to AITC 119 ADV's for Hardwood Glulam Members

As a result of the recent research conducted at Penn State University; West Virginia University; and the Forest Products Lab in Madison, Wisconsin, additional lamination layup combinations for hardwood glulam members are under development by AITC. These layup combinations and associated ADV's should be published by AITC in the near future. Some of the hardwood lamination combinations under consideration are listed in Table 1.5 and sketched in Figure 1.9.

Table 1.5. Proposed Hardwood Lamination Combinations.

Combination[1]	Species	Lumber Grade in Listed Portion of Cross Section				
		Outer Tension	Next Inner	Core	Next Inner	Outer Compression
2400f-1.8E	RM or YP	10%[2] 2.0-1/6[3]	15% 1.8-1/3	50% No. 2	15% 1.8-1/3	10% 2.0-1/3
2000f-1.6E	RM or YP	15% 1.8-1/6	10% 1.5-1/3	50% No. 2	10% 1.5-1/3	15% 1.8-1/3
2000f-1.6E	NRO	25% Sel. Struct.	--	50% No. 2	--	25% Sel. Struct.
1600f-1.4E	RM, YP or NRO	--------------------All No. 2 Visually Graded--------------------				

1 Combination defines F_{bx} and MOE of section.
2 Percentage of cross section.
3 2.0-1/6 MOE--maximum portion of lamination face with knot or slope of grain deviation.

1.3.4 Recommended ADV's for Hardwood Glulam Bridge Girders and Bridge Deck Panels

ADV's recommended and subsequently used in the PennDOT BLC-560 Series, "Standard Designs for Hardwood Glulam Timber Bridges," for bridge girders and bridge decks are listed in Tables 1.6 and 1.7. ADV's are listed in Table 1.6 for northern red oak (NRO) Combination A lamination layups (Figure 1.8a) and for red maple (RM) or yellow poplar (YP) Combination 2400f-1.8E lamination layups (See Figure 1.8b and Table 1.5 for typical designs). The values in Table 1.6 are for normal load duration, dry use (m.c. $\leq$16%), loads applied normal to the lamination faces, and standard beam sizes (12-inch deep section for NRO and 5.125-inch wide by 12-inches deep by 21-feet long beam for RM and YP).

ADV's listed in Table 1.7 are for deck panels manufactured with visually graded No. 1 or No. 2 NRO, RM, or YP throughout. The values listed in Table 1.7 are for normal load duration, dry use (m.c. $\leq$ 16%), loads applied parallel to the lamination faces, and decks nominally 4 to 6 inches deep.

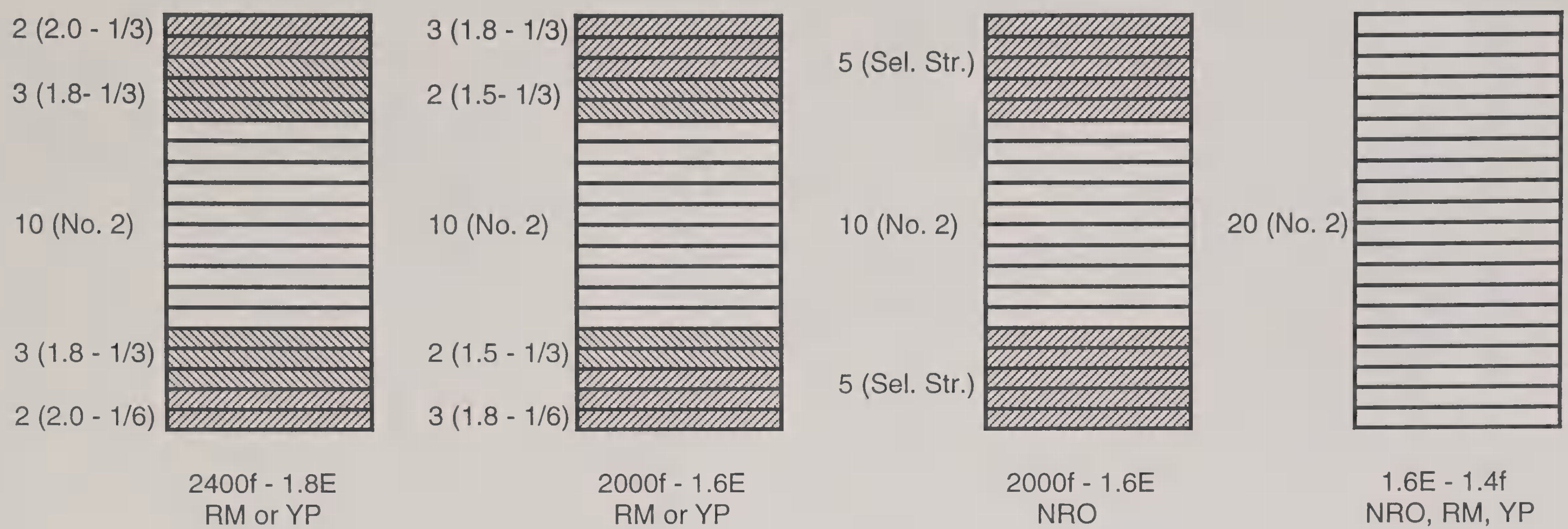


Figure 1.9 Proposed hardwood glulam layups.

Table 1.6. Recommended Dry Use ADV's for NRO, RM, and YP Glulam Bridge Girders (psi)[a]

Property	Species		
	NRO[b]	RM[c]	YP[c]
F_{bx}	2400	2400	2400
E_x	1.8×10^6	1.8×10^6	1.8×10^6
F_v	215	205	145
$F_{c\perp}$	885	615	420

[a] Dry use (m.c. $\leq$ 16%) and normal load duration
[b] Combination A NRO layup; d $\leq$ 12 in.
[c] Combination 2400 f-1.8E RM or YP layup; normalized to b x w x L = 5.125 in. x 12 in. x 21 ft.

Table 1.7. Recommended Dry Use ADV's for NRO, RM, and YP Glulam Bridge Deck Panels (psi)[a]

Property	Species[b]		
	NRO	RM	YP
F_{by}	1800	1800	1400
E_y	1.6×10^6	1.7×10^6	1.4×10^6
F_v	215	205	145
$F_{c\perp}$	885	615	420

[a] For 4- to 6-inch thick decks, normal duration loads and m.c.$\leq$ 16%.
[b] Laminations all No. 1 or No. 2 VSR lumber

1.3.5 Adjustments to the ADV's for Hardwood Glulam Bridge Girders and Deck Panels

1.3.5.1 Wet Use Factors, C_m

The ADV's of glulam decrease as moisture content (m.c.) increases. Thus, if the moisture content of the beams and girders exceeds 16%, the ADV's in Tables 1.6 and 1.7 have to be reduced by the wet use factors listed in Table 1.8.

Table 1.8. Wet Use Factors C_m, [a]

Property	C_m
F_{bx}, F_{by}	0.800
E_x, E_y	0.833
F_v	0.875
$F_{c\perp}$	0.530

[a] Table 5C NDS® Supplement

Gutkowski and McCutcheon (1987), based on moisture content measurements of numerous timber bridges, argue that it is acceptable to use dry use ADV's for glulam girders under glulam decks with the exception of F_{cp} (bearing) strength. However, they recommend use of wet use ADV's for the deck panels. The PennDOT BLC-560 Series Standard Designs, (PennDOT, 1994) which are based on wet use ADV's throughout for all girders and deck panels, take a more conservative approach.

1.3.5.2 Volume (Size) Effect Factor, C_v

The F_{bx} values in Table 1.6 are for standard size beams. For different size beams, F_{bx} must be adjusted by the factor C_v defined in equations 10 or 11.

For NRO beams:

$$C_v = (12/d)^{1/9} \tag{10}$$

For RM or YP beams:

$$C_v = \left(\frac{12}{d}\right)^{0.10} \left(\frac{5.125}{b}\right)^{0.10} \left(\frac{21}{L}\right)^{0.10} \tag{11}$$

where d = depth, in.
b = width, in.
L = length, ft.

1.3.6 Final Design Values for Bridges

The final design values for bridge girders and deck panels are the ADV's in Tables 1.6 or 1.7 multiplied by the appropriate wet use factor and volume factor.

$$F_{bx}' = C_m C_v F_{bx} \tag{12a}$$
$$F_{by}' = C_m F_{by} \tag{12b}$$
$$E_x' = C_m E_x \tag{12c}$$
$$E_y' = C_m E_y \tag{12d}$$
$$F_v' = C_m F_v \tag{12e}$$
$$F_{c\perp}' = C_m F_{c\perp} \tag{12f}$$

1.4 How to Specify and Design with Hardwood Glulam

The designer uses the same principles of structural analysis and design as would be used for other materials, especially softwood species of glulam timbers. The designer doesn't have to design the lamination layups. It is sufficient to:

- Determine the hardwood species and combination or allowable stresses desired or required for the specific project.

- Select the appropriate sizes required for the application, taking into account conditions of use, duration of load, size factor, and other factors following good engineering practice.

- Specify the lamination layup combination, species, sizes, and special fabrication details.

A detailed example of this procedure is outlined in Section 2.

Section 2: Hardwood Glulam Bridge Design

2.1 Part I: Design Criteria

2.1.1 Specifications and References

1. "Standard Specifications for Highway Bridges," 15th Edition, 1992; Adopted by the American Association of State and Highway Transportation Officials (AASHTO).

2. "Design Manual Part 4; Structures." (DM 4) Pennsylvania Department of Transportation (PennDOT).

3. "Standards for Hardwood Glulam Timber Bridge Design." (BLC-560 Series) Pennsylvania Department of Transportation (PennDOT).

4. "Timber Bridges; Design, Construction, Inspection, and Maintenance." Michael A. Ritter, United States Department of Agriculture Forest Service.

2.1.2 Design Live Loads (AASHTO 3.7; PennDOT DM 4)

Timber beam and deck design procedures are limited to AASHTO Group 1 loads where the design is controlled by a combination of structure dead load and vehicle live load. For many states, vehicle live loads for timber bridges are the standard AASHTO loads consisting of H15-44, H20-44, HS15-44, and HS20-44 and alternate military load. However, in some states, such as Pennsylvania, the vehicle live loads are increased to HS25 and increased military load (1.25 x HS20 and 1.25 x alternate military load) for state and federally funded bridges.

Also, the maximum legal load permitted on some state highways has recently increased. Special loads, such as the ML80 4-axle truck, should be considered during design. Figure 2.1 compares the maximum simple-span bending moments produced by the load configurations previously discussed above, expressed as a percentage of the HS-20 Load.

For deck design, AASHTO special provisions for H20-44 and HS20-44 loads apply, and a 12,000 pound wheel load is used. In Pennsylvania, although not specified, it is suggested that this load be increased by 25% to 15,000 pounds to be consistent with the heavier allowable vehicle loads.

Dead Loads

The dead load of creosote hardwood glulam timbers is generally 50 pcf. However, some species are significantly more dense than others, and this should be considered in the design. As an aid, the unit weight of the three approved hardwood glulam species is as follows:

Northern Red Oak	=	55 pcf	
Red Maple	=	50 pcf	(Ref. PennDOT BLC 560)
Yellow Poplar	=	50 pcf	

In most cases, deck design will include a 3" bituminous wearing surface and a 2.75" future wearing surface.

3" wearing surface	=	37.5 psf
2.75" future wearing surface	=	35.0 psf

2.1.3 Impact (AASHTO 3.8 and 13.3)

Impact is not included for timber structures.

2.1.4 Live Load Deflection (PennDOT DM-4 PP3.5.12)

Flexural members shall be designed so that deflection due to service live loads shall not exceed L/500 of the span.

2.1.5 Live Load Distribution (AASHTO 3.23)

For interior stringers, the wheel load distribution factors listed in Table 3.23.1 of AASHTO for glulam timber panels on glulam stringers shall be used to calculate the moment. The deflection produced by one wheel load shall be distributed equally to one half the stringers in the lane.

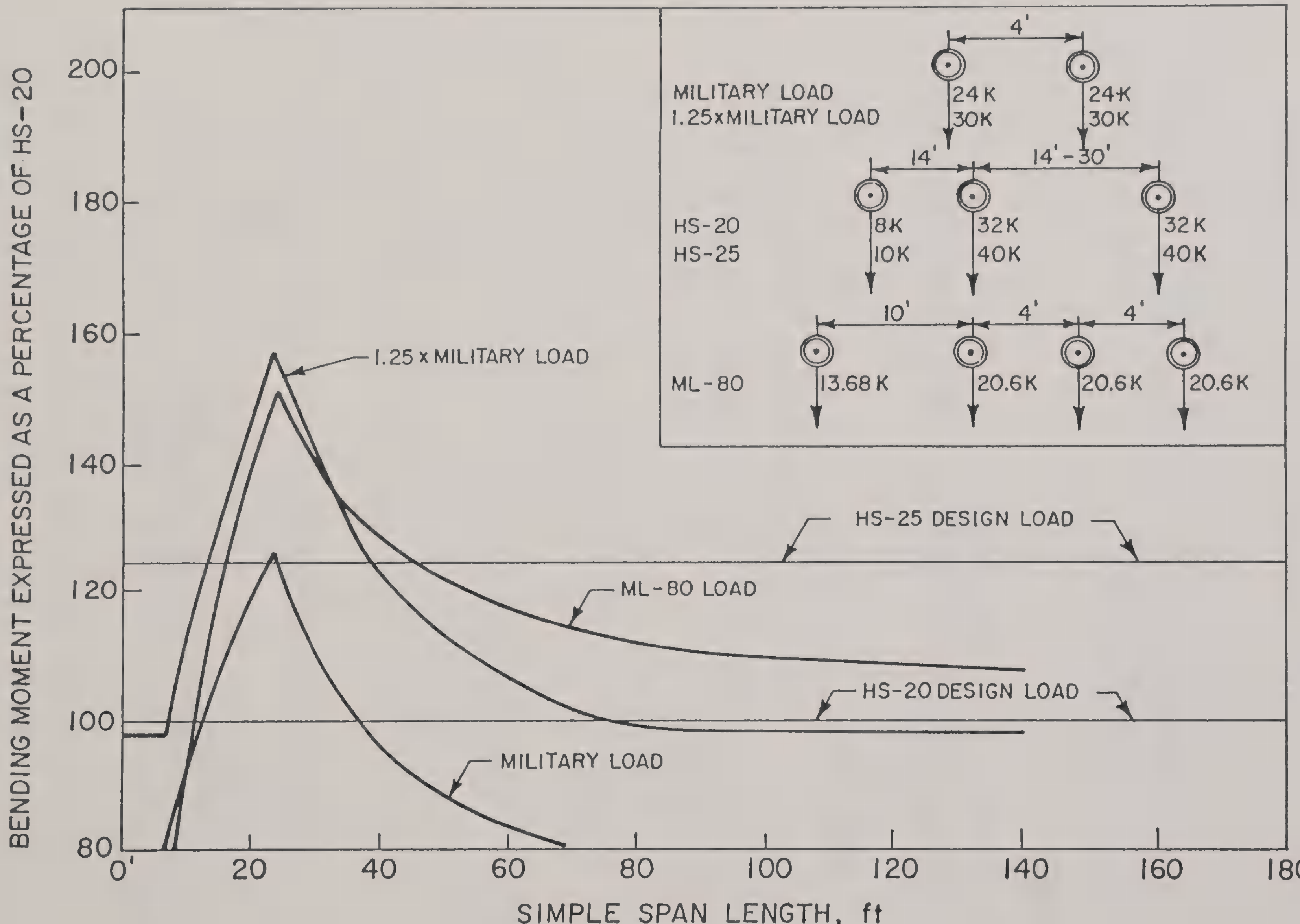


Figure 2.1 Comparison of maximum simple span bending moments. (Figure C.3.76(A) in PennDOT DM-4 Commentary, 1992)

Table 2.1. Moment and Deflection Distribution Factors

Nominal Deck Thickness	Moment (WL/BM)		Deflection (WL/BM)	
	1 Lane	2 Lanes +	1 Lane	2 Lanes +
4	S/4.5	S/4.0	2/#BMS	(2 x #lanes)/#BMS
$\geq$6	S/6.0	S/5.0	2/#BMS	(2 x #lanes)/#BMS

Note: If S exceeds 6.0 ft. for one lane and 7.5 ft. for two or more lanes, the moment will be distributed assuming the deck between beams acts as a simple beam.

S = average beam spacing (ft.)

For exterior stringers the moment shall be distributed, assuming the deck acts as a simple span between stringers.

2.1.6 Shear Distribution: (AASHTO 13.6.5)

The maximum horizontal shear, due to vehicle live loads, for glulam beams is calculated at a distance from the support equal to 3 times the depth of the beam (3d) or at the span quarter point (L/4), whichever is less. The live load shall be placed to cause maximum shear at this location. The shear shall be calculated as one half the sum of 60 percent of the shear from the undistributed wheel load and the shear from wheel loads distributed laterally as specified for moment (AASHTO 3.23). One line of wheels is assumed to be carried by one beam for the undistributed wheel loads.

2.1.7 Reaction Distribution: (AASHTO 3.23)

Lateral distribution of the wheel loads for calculation of the reaction shall be that produced by assuming the flooring to act as a simple span between stringers.

2.1.8 Wheel Load Distribution on Decks: (AASHTO 3.25.1.1)

In the direction of the deck panel span, the wheel load shall be distributed over the width of tire, (b_t), as calculated by:

$$b_t = \sqrt{0.025P}$$

where B_t = Wheel load distribution width (in.)
P = Maximum wheel load (12,000 lbs, for HS-20)
(15,000 lbs. for HS-25)

Normal to the direction of the span, for non-interconnected glulam decks, the wheel load is distributed over an effective width, b_d, equal to the deck thickness, t, plus 15 inches, but not to exceed panel width.

$b_d = 15 + t \leq$ panel thickness

Normal to the direction of the span, for interconnected glulam decks (not less than 6 inches thick), with adequate shear transfer between panels, such as by using dowels along the panel joint, the wheel load is distributed over an area, b_d, equal to twice the deck thickness, t, plus 15 inches, not to exceed panel width.

$b_d = 15 + t \leq$ panel thickness

Table 2.2. Material Properties and Adjustment Factors (PennDOT BLC 560)

Allowable stresses:

Property	Northern Red Oak Combination A	Red Maple 24F-1.8E	Yellow Poplar 24F-1.8F	Wet-use factor* (CM)
Stringers:				
F_b	2400	2400	2400	0.800*
F_v	215	205	145	0.875
$F_{c\perp}$	885	615	420	0.530
E_x	1.8×10^6	1.8×10^6	1.8×10^6	0.833
Decks:				
F_{by}	1800	1800	1400	0.800
F_v	215	205	145	0.875
E_y	1.6×10^6	1.7×10^6	1.4×10^6	0.833

* Pennsylvania requires the application of the wet-use factor to the allowable properties of all bridge components.

A size or volume factor (C_f or C_v) shall be applied to the stringers in accordance with the following:

Northern Red Oak: $C_f = (12/d)^{1/9}$
Red Maple and Yellow Poplar: $C_v = (5.125/b)^x \bullet (12/d)^x \bullet (21/L)^x$

where: b = beam width, inches
d = beam depth, inches
L = beam length, feet
x = 0.071 red maple; 0.088 yellow poplar; 0.10 conservative value for hardwoods in general

2.2 Part 2: Design Procedures

2.2.1 Beam Design Procedures

2.2.1.1 Define Basic Geometry and Criteria:

- Span length (L) (center to center bearings)
- Roadway width and deck width (out to out)
- Number of lanes

- Number and spacing of beams
- Assumed deck thickness
- Determine Distribution Factors, DF

2.2.1.2 Select Beam Species and Allowable Properties:

- Northern Red Oak; Combination A
- Red Maple; 24F-1.8E
- Yellow Poplar; 24F-1.8E

2.2.1.3 Determine Deck Dead Loads and Dead Load Moment (M_{DL}):

- Use uniformly distributed load relative to the tributary width curned by each beam
- Equally distribute railing and parapet load to each beam

2.2.1.4 Compute Live Load Moment (M_{LL}):

- $M_{LL} / B_M = M_{LL} / WL \bullet DF$

2.2.1.5 Determine Beam Size and Required Section Modulus Based on Bending:

$$S_x(C_f \text{ or } C_v) = \left(\frac{M_{LL} + M_{DL1}}{F_{bx} \bullet C_m}\right)$$

Add beam dead load moment (M_{DL2}) and check

$$f_b = \frac{M_{LL} + M_{DL1} + M_{DL2}}{S_x}$$ where: S_x = section modulus

$$f_b \leq Fb_x \bullet (C_f \text{ or } C_v) \bullet C_m$$ $C_m = 0.800$

2.2.1.6 Check Live Load Deflection:

$\Delta_{LL} \leq L/500$ where: L = span length

2.2.1.7 Check Horizontal Shear:

$$f_v = \frac{3 \bullet V}{2 \bullet A}$$

$$f_v \leq F_v \bullet C_m$$

where: V = maximum live load and dead load shear
A = effective cross sectional area
$C_m = 0.87$

2.2.1.8 Check Other Loading Conditions:

- Overloads (P-82)
- Lateral loads (wind, seismic, ice)
- Longitudinal and centrifugal forces

Stresses from other loading combinations (AASHTO; Table 3.22.1A) may be modified by adjustments for duration of load when applicable.

2.2.1.9 Determine Bearing Area and Stress

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{A_b}$$ where: R_{DL} = Dead load reaction

$$f_{c\perp} \leq F_{c\perp} \bullet C_m$$

R_{LL} = Live load reaction

A_b = Bearing area

C_m = 0.530

2.2.1.10 Determine Beam Camber:

Fabricated beam camber shall be equal to 3 times the total dead load deflection.

$$\text{Camber} = 3 \bullet (\Delta_{\text{deck DL}} + \Delta_{\text{beam DL}})$$

For spans less than 50 feet, the camber calculated as above may be too small for practical fabrication; therefore, a 1" minimum camber is recommended.

2.2.2 Deck Design Procedures

2.2.2.1 Non-Interconnected Deck Design

The deck is assumed to act as a simple span between beams.

2.2.2.1.1 Define Effective Deck Span and Wheel Loads and Estimate Thickness:

The deck span, S, is the clear distance between supporting beams plus one-half the beam width, but not greater than the clear span plus the panel thickness (AASHTO 3.25.1.2).

Hardwood glulam decks are fabricated in two common thicknesses 3.125 inches and 5.125 inches (actual).

2.2.2.1.2 Determine Effective Deck Section Properties:

Area $(A) = b_d \bullet t$ (in.^2)

Section Modulus $(S_y) = \dfrac{b_d \bullet t^2}{6}$ (in.^3)

Moment of Inertia $(I_y) = \dfrac{b_d \bullet t^3}{12}$ (in.^4)

2.2.2.1.3 Calculate Dead Load and Live Load Moments, Shear, and Deflection:

For a 12,000 lb. wheel load distributed over a width of 17.32 inches, the following equations apply:

$$M_{DL} = \frac{W_{DL} \bullet S^2}{8} \quad \text{(in.-lb.)}$$

where: W_{DL} = Deck and wearing surface over b_d. (lb./in.)
$S > 17.32$ in.
$S \leq 122$ in. for monemt
$S \leq 110$ in. for deflection

$$M_{LL} = (3000 \bullet S) - 25983 \quad \text{(in.-lb.)}$$

$$V_{DL} = W_{DL} \bullet (\frac{S}{2} - t) \quad \text{(lb.)}$$

V_{LL} = maximum shear when the edge of the wheel load distribution width is placed a distance t from the support

$$\Delta_{LL} = \frac{1.80}{E_y I_y}(138.8 \bullet S^3 - 20,780 \bullet S + 90,000) \quad \text{(in.)}$$

where: E_y = modulus of elasticity about the y-axis (wet use value).

Note: When the deck is continuous over more than two spans, the moments and live load deflections are multiplied by a factor of 0.80.

2.2.2.1.4 Check the Stresses and Deflections Against the Allowables:

Same procedure as for beam design previously discussed, except recommended live load deflection is 0.10 in. (USDA, Timber Bridges)

2.2.2.1.5 Check Overhang:

- Assume deck overhang is measured from centerline of outside beam minus 1/4 of the beam width.

- Centerline of wheel load is placed at 1'-0" from face of curb or railing.

2.2.2.2 Doweled Deck Design

2.2.2.2.1 Define Deck Span and Effective Properties
and 2 Same procedure as non-interconnected deck design previously discussed. However, the wheel loads from the AASHTO special provisions do not apply.

2.2.2.2.3 **Calculate primary dead load moment and shear**, (same as non-interconnected).

2.2.2.2.4 Determine Primary Live Load Moment & Shear: (AASHTO 3.25.1.3)

$$M_x = P[(0.51 \bullet \text{Log}_{10} \bullet S) - K]$$

$$R_x = 0.034 \bullet P$$

where:
M_x = Primary live load moment (in.-lb./in.)
R_x = Primary live load shear (lb./in.)
P = Design wheel load (20,000 lb. for HS25)
K = Design constant (0.51 for HS20 & HS25)
S = Effective deck span (in.)

2.2.2.2.5 Select Allowable Properties & Required Deck Thickness: (AASHTO 3.25.1)

$$t = \sqrt{\frac{6 \bullet (M_x + M_{DLx})}{F_b'}}$$

or

$$t = \frac{3 \bullet (R_x + M_{DLx})}{2 \bullet F_v'}$$

where: $F_b' = F_{by} \bullet C_m \bullet C_f$

$F_v' = F_{vy} \bullet C_m$

Use the larger (AASHTO 3.25.1.3)
Note: When the deck is continuous over more than two spans, the moments are 80% of the simple span value.

2.2.2.2.6 Check Live Load Deflection:
(USDA, Timber Bridges, EQ 7-34)

$\Delta_{LL} \leq 0.10$in. where: $\Delta_{LL} = \dfrac{0.51\,P \bullet S \bullet (S - 10)}{E_y\ t^3}$

Note: When deck is continous over more than two spans, the deflections are 80% of the calculated value.

2.2.2.2.7 Compute Secondary Moment and Shear:
(AASHTO 3.25.1.4)

$$M_y = \frac{P \bullet S}{1600}(S - 10)$$

$$R_y = \frac{6 \bullet P \bullet S}{1000}$$

When effective span length, S, is ≤ 50 inches.

2.2.2.2.8 Determine Required Size and Spacing of Dowels:

$$n = \frac{1000}{\sigma_{pl}}\left(\frac{R_y}{R_o} + \frac{M_y}{M_o}\right)$$

where: n = number of dowels for each span

σ_{pl} = Proportional limit stress for timber
Northern Red Oak = 1010 psi
Red Maple = 1000 psi
Yellow Poplar = 500 psi

R_o = Dowell shear capacity (AASHTO 3.25.1.4)

M_o = Dowell moment capacity

2.2.2.2.9 Check Overhang:

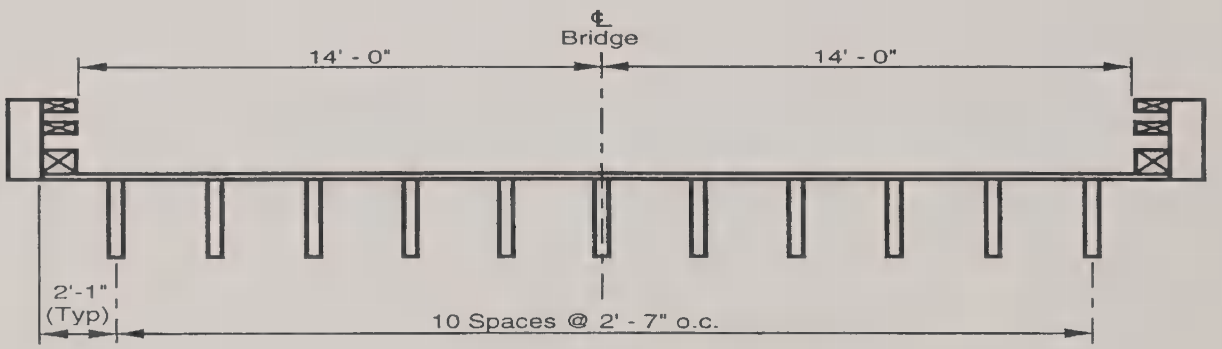
Same as non-interconnected deck.

Section 3: Hardwood Glulam Bridge Design Example

3.1 Define Base Geometry

Given Parameters:

- Two-lane highway bridge
- Placed on existing abutment
- Span length between center lines of bearing = 30'-0"
- Road width = 28'-0"
- Out-to-out deck = 30'-0"
- Number of stringers = 11
- Stringer spacing, s, = 2'-7"
- Assumed deck thickness = 4 inches
- Cross-section as shown



3.2 Select Species and Allowable Properties

- Species = Northern Red Oak
- Allowable Properties for Combination A

F_{bx} = 2400 psi
F_v = 215 psi
E_x = 1.8×10^6 psi
$F_{c\perp}$ = 885 psi

3.3 Determine Dead Loads

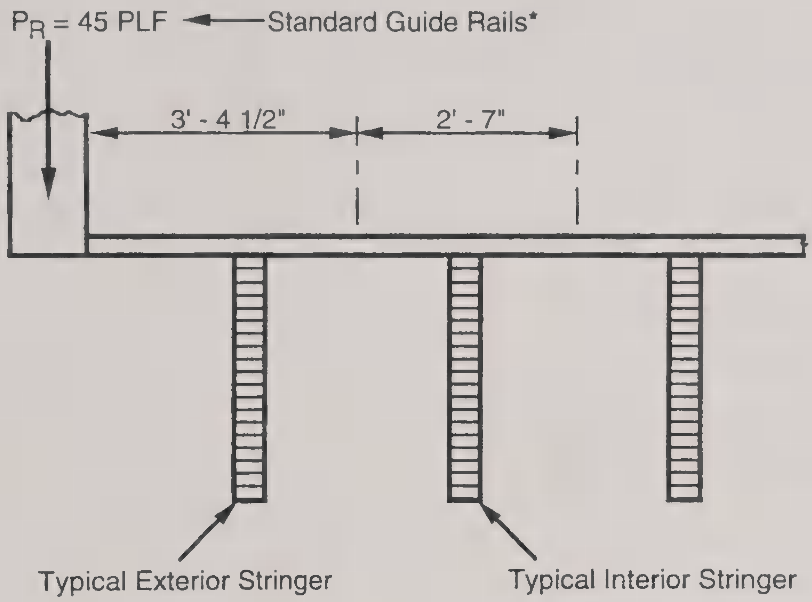
Assume all timber components are pressure treated with creosote to a retention level of 12 pcf in the treated zone.

Creosote Treated Northern Red Oak Unit Weight	= 55 pcf	BLC-560
Bituminous Wearing Surface	= 37.5 psf	AASHTO 3.3
Future Wearing Surface	= 35.0 psf	

$$\text{Deck Dead Load} = \frac{4\text{in.} \times 55\# \ / \text{ft.}^3}{12\text{in.}/\text{ft.}} = 18.33 \text{ psf}$$

Total Deck DL = 90.83 psf

The tributary deck width for typical interior stringer is 2.58 feet. The tributary width for the exterior stringer is 3.37 feet.



P_R = Rail and Curb Load: Assume 6'-0" post spacing

*Note: Some railing systems can produce a significant dead load. For example, the glulam system specified by PennDOT (see BLC-560) weighs approximately 120 PLF (see below).

Scuppers – 5 - 6" x 12" x 3'/30' x 55 pcf	=	13.8 plf
Posts – 5 - 10" x 12" x 2.75'/30' x 55 pcf	=	21.0 plf
Curb and Rails – 3 - 6" x 12" x 55 pcf	=	82.5 plf
TOTAL	=	117 plf
Use DL	=	120 plf

3.4 Determine Dead Load Moments

For interior beams:

Deck W_{DL} = 2.58 ft . x 90.8 psf = 234.2 plf

$$M_{DL1} = \frac{234.2 \text{ plf x } (30\text{ft.})^2}{8} = \underline{26{,}335\text{ft.-lb.}}$$

For exterior beams:

Deck W_{DL} = 3.37 ft . x 90.8 + 45 = 351 plf

$$M_{DL1} = \frac{251 \text{ plf x } (30\text{ft.})^2}{8} = \underline{39{,}487\text{ft.-lb.}}$$

3.5 Determine Live Load Moments

Maximum wheel line moments for 30 ft. span

HS20	=	141.07 ft-kips	AASHTO Appendix
HS25	=	176.34 ft-kips	1.25 x HS-20
Governs IML	=	196.00 ft-kips	PC-Bridge
ML80	=	195.95 ft-kips	PC-Bridge

HS20 and ML80 included for information purposes only. They are not design vehicles for state highways. — PennDOT DM4, 2.4

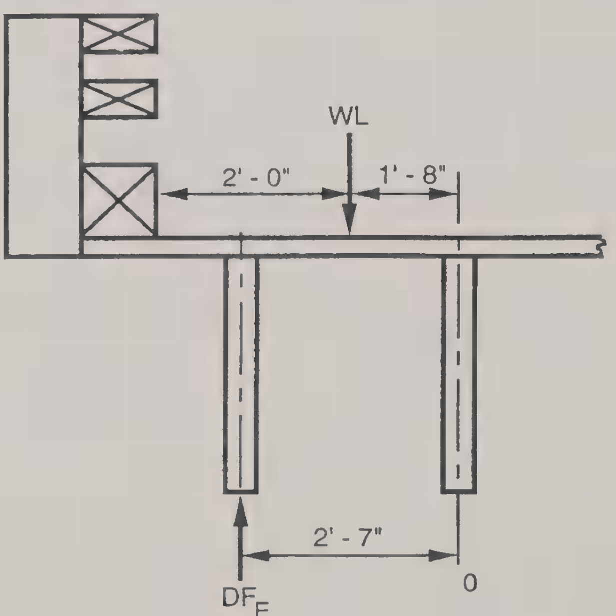
Distribution factors (DF):

For interior beams

DF = S/4.0 (For 4-inch decks)

<u>DF = 2.58/4.0 = 0.646</u>

For exterior beams, assume flooring to act as a simple span between beams. The wheel load is placed 2'-0" from face of curb. — AASHTO Table3.23.2



Summing Moments About 0:

$$DF_E = \frac{1.67'}{2.58'} = 0.645$$

However, in no case shall an exterior stringer have less carrying capacity than an interior stringer. — AASHTO 3.23.2.3

$\therefore DF_I = DF_E = 0.646$ WL/BEAM

Live load moment: (Interior = Exterior)

M_{LL} = 196.0 ft-kips x 1000 x 0.646
= 126,616 ft-lbs

3.6 Determine Beam Size

For northern red oak, the allowable bending stress, F_{bx}, is modified by the wet-use factor, $C_m = 0.80$, and by the size factor, $C_f = (12/d)^{1/9}$. NDS 1991 and BLC-560

$F_b' = F_{bx} C_m C_f = 2400$ psi (0.80) C_f
$F_b' = 1920 \bullet C_f$ psi

Total Moments:

Interior Beams

$M_{TI} = 26{,}355 + 126{,}616 = 152{,}971$ ft.-lbs.

Exterior Beams

$M_{TE} = 39{,}487 + 126{,}616 = 166{,}103$ ft.-lbs. (Governs)

Required Section Modulus and Beam Depth

$$S_x\, C_f = \frac{M}{F_b'} = \frac{166{,}103 \bullet 12 \text{ in./ft.}}{1920 \text{ psi}} = 10.38 \text{ in.}^3$$

Assume beam width = 5 1/8"

Solve for depth:

$$S_x\, C_f = \frac{b \bullet d^2}{6}$$

$$\therefore d = \sqrt{\frac{S_x \bullet C_f \bullet 6}{b}} = \sqrt{\frac{10.38 \text{ in.}^3 \bullet 6}{5.125}}$$

d = 34.8 in.

Adjust moments and allowable stresses for beam self weight and size factor.

Required d = 34.8"; actual depth based on 1.5 in. laminations:

Number of 1.5 in. lams = 34.8/1.5 = 23.2

Say 24 @ 1.5 in. = 36 in.; $C_f = (12/36)^{1/9} = 0.88$

Thus, try 25 @ 1.5 in. = 37.5 in.

Unit Weight = 55 pcf (N. Red Oak)

$$W_{DL} = \frac{55 \text{ lb./ft.}^3 \bullet 5.125 \text{ in.} \bullet 37.5 \text{ in.}}{144 \text{ in.}^2/\text{ft.}^2} = 73.4 \text{ plf}$$

$$M_{DL\,2} = \frac{73.4 \text{ plf} \bullet (30 \text{ ft.})^2}{8} = 8{,}258 \text{ ft.-lb.}$$

Check stresses:

$$S_x = \frac{5.125 \bullet 37.5^2}{6} = 1201 \text{ in.}^3$$

$$C_V = (12/37.5)^{1.9 \text{ or } 0.11} = 0.88$$

Note: Equation 10 in Section 1.3.5 was used here because the glulam beams are Northern Red Oak. If they were Red maple or yellow poplar, Equation 11 would have been used.

$$f_b = \frac{M_T}{S_x} = \frac{(166{,}103 + 8{,}258) \bullet 12}{1201} = \underline{1742} \text{ psi}$$

$$F_b' = 2400 \bullet 0.80 \bullet 0.88 = \underline{1690} \text{ psi}$$

$f_b >> F_b'$

1742 >> 1690 N.G. (No Good or inadequate)

$\therefore$ Add Depth and recheck

Assume $C_F = 0.87$ (conservative)

$$\text{Ratio } \frac{f_b}{F_b'} = \frac{1742}{1690} = 1.03$$

New $S_x = 1201 \bullet 1.03 = 1237 \text{ in}^3$

$$\text{Required } d = \sqrt{\frac{1237 \bullet 6}{5.125}} = 38.0 \text{ in.}$$

26 - 1.5 in. laminations = 39 in.; thus use d = 39 in.

Check:

$$C_F = (12/39)^{1/9} = 0.877 > 0.87$$

$$M_{DL2} = \frac{55 \bullet 5.125 \bullet 39 \bullet 30^2}{144 \bullet 8} = 8588 \text{ ft.-lb.}$$

$$f_b = \frac{166{,}103 \ + \ 8{,}588 \bullet 12}{5.125 \ \bullet \ 39^2 / 6} = 1613 \text{ psi}$$

$F_b' = 2{,}400 \times .80 \times 0.877 = 1684$
$f_b << F_b'$
1613 << 1684, thus section is okay.

Note: Assume the deck is fastened to the stringers with lag bolts at 1'-0" o.c. along the length of the beam. Therefore, lateral support is continuous along the top, and no reduction is required for slenderness. AASHTO 13.3.8.2.2

Also a 3-1/8" wide by approximately 34.5" deep diaphragm will be placed at the center line of the span between each girder and at each end.

3.7 Check Live Load Deflection

The deflection coefficients in lb.-in.^{3} for the design vehicle wheel lines (WL) are:

HS25 $= 2.82 \times 10^{10}$ (lb.-in.^{3}) PC Bridge Program
IML $= 2.84 \times 10^{10}$ (lb.-in.^{3}) (Governs)

Deflection coefficient = $\Delta_{LL} \bullet Ex \bullet I$

$$\therefore \Delta_{LL} = \frac{\text{Deflection coefficient (lb.-in.}^3\text{)}}{E_x' \bullet I \text{ (lb.-in.}^2\text{)}}$$

$$E_x' = E_x C_m$$

where: $C_m = 0.833$

$$E_x' = 1.8 \times 10^6 \bullet 0.833 = 1.499 \times 10^6 \text{ psi}$$

$$I = \frac{5.125 \bullet 39^3}{12} = 25{,}334 \text{ in.}^4$$

$$\therefore \ \Delta_{LL} / WL = \frac{2.84 \times 10^{10}}{1.499 \times 10^6 \bullet 25{,}334} = 0.747 \text{ in./WL}$$

Distribute deflection evenly to one-half the beams in the lane

$DF = 2 \times 2$ lanes/11 beams = 0.364 WL/BM

$\Delta_{LL}/WL = 0.747$ in./WL • 0.364 WL/beam = 0.272 in./beam

Thus, $L/\Delta_{LL} = 30 \times 12/0.272 = 1324 >> 500$ (Therefore adequate)

3.8 Check Beam Horizontal Shear

The vertical shear for the design vehicle wheel loads is:

Live Load Shear:

HS25 = 30.25 kips (Governs)
IML = 27.55 kips

PC Bridge Programs

Dead Load Shear:

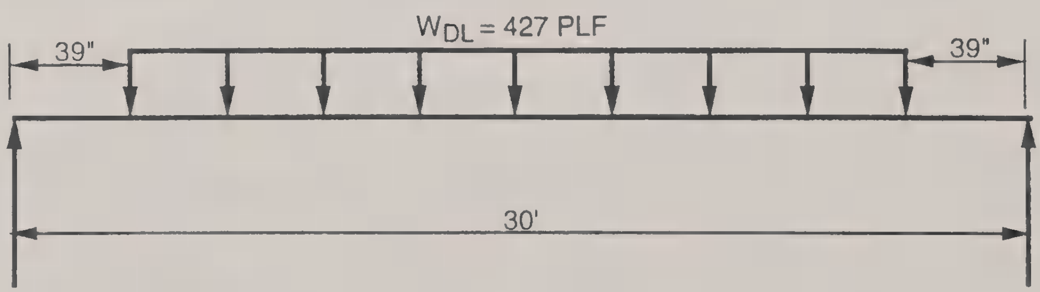
From bending calculations

W_{DL1} (deck and railing) = 351 plf

W_{DL2} (beam) = 76 plf

Total W_{DL} = 427 plf

Calculate shear at distance d from support. AASHTO 13.3



$$V_{DL} = W_{DL}\left(\frac{L}{2} - d\right)$$

$$V_{DL} = 427\left(\frac{30}{2} - \frac{39}{12}\right) = 5017 \text{ lbs.}$$

Distribution of V_{LL} to beams: AASHTO 13.3

$$V_{LL} = 0.5\ [0.6\ V_{LU} + V_{LD}]$$

where: V_{LU} = undistributed load
V_{LD} = distributed same as moment

$$V_{LU} = 30{,}250 \text{ lbs.}$$

$$V_{LD} = 30{,}250\ (0.646) = 19{,}542 \text{ lbs.}$$

$$V_{LL} = 0.5\ [0.6 \bullet 30{,}250 + 19{,}542] = 18{,}846 \text{ lbs.}$$

Find shear stress:

$$V_{TOTAL} = V_{LL} + V_{DL} = 18{,}846 + 5{,}017 = 23{,}863 \text{ lbs.}$$

$$f_v = \frac{3\ V_{TOTAL}}{2\ A} = \frac{3 \times 23{,}863}{2 \times 5.125 \bullet 39}$$

$$f_v = 179 \text{ psi}$$

$$F_v' = F_v \bullet C_m = 215 \bullet 0.875 = \underline{188} \text{ psi} > f_v = 179 \text{ psi}$$

Thus section adequate for shear.

3.9 Check Other Loading Conditions

Ratings and other group loadings are not covered here.

3.10 Determine Beam Bearing Area

Assume wheel line directly over stringer; thus distribution factor (DF) = 1.0

DL-reaction = 427 plf • 30 ft. • 0.5 = 6,406 lb.

LL-reaction = (WL • 1.0) = <u>31,000</u> lb.

Total Reaction = 37,406 lb.

$$F_{c\perp}' = F_{c\perp} \bullet C_m = 885 \bullet 0.533 = 472 \text{ psi}$$

$$\text{Required Bearing Length} = \frac{37{,}400}{5.125 \bullet 472} = 15.5 \text{ in.}$$

3.11 Determine Beam Camber

Fabricated camber shall be greater than or equal to 3 times the total dead load deflection. BLC-560

$$\Delta_{DL} = \frac{5 \bullet W_{DL} \bullet L^4}{384 \bullet E'x \bullet I}$$

$$W_{DL} = 427 \text{ PLF}$$

$$E'x = 1.499 \times 10^6 \text{ psi}$$

$$I = 25{,}334 \text{ in}^4$$

$$\Delta_{DL} = \frac{5 \bullet 427 \bullet 30^4 \bullet 1728}{384 \bullet 1.499 \times 10^6 \bullet 25{,}334}$$

$$\Delta_{DL} = 0.221$$

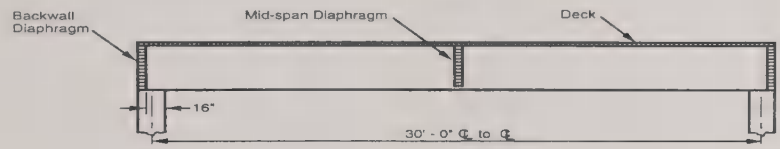
Camber = 3 x 0.221 = 0.66 in.

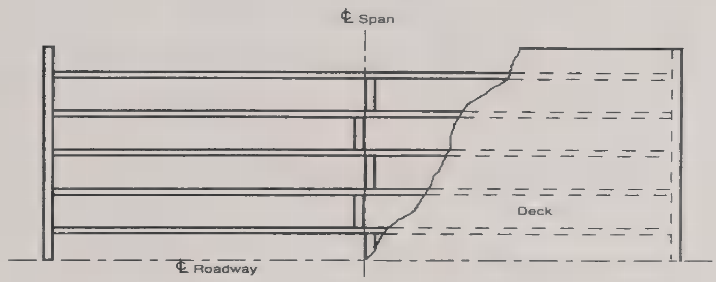
Use Camber = 1.0 in. for ease of fabrication

3.12 Beam Summary

Stringers: 11 - 5 1/8" x 39" x 31' - 4" Spaced at 2' - 7" o.c.

Diaphragms: 10 - 3 1/8" x 34.5" x 2.156' @ center span





3.13 Deck Design

3.13.1 Define Effective Deck Span(s) and Loads

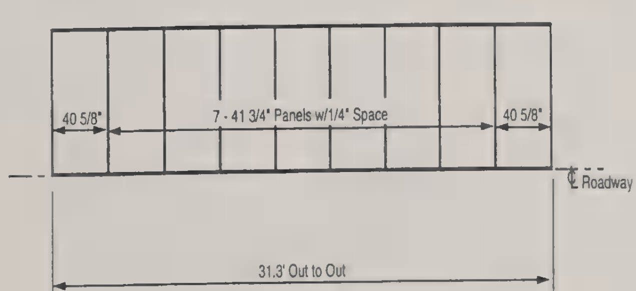
Assume panel thickness equals 4".

Clear distance = 31 in. - 5.125 = 25.875 in.

$$S = 25.875 + \frac{5.125}{2} = 28.44 \text{ in.}$$

Check 25.875 + 4 = 29.875 > 28.44 AASHTO 3.25.1.2

For ASSHTO HS25 truck using special provisions for timber flooring, the design wheel load is 12,000 lb. x 1.25 = 15,000 lb.



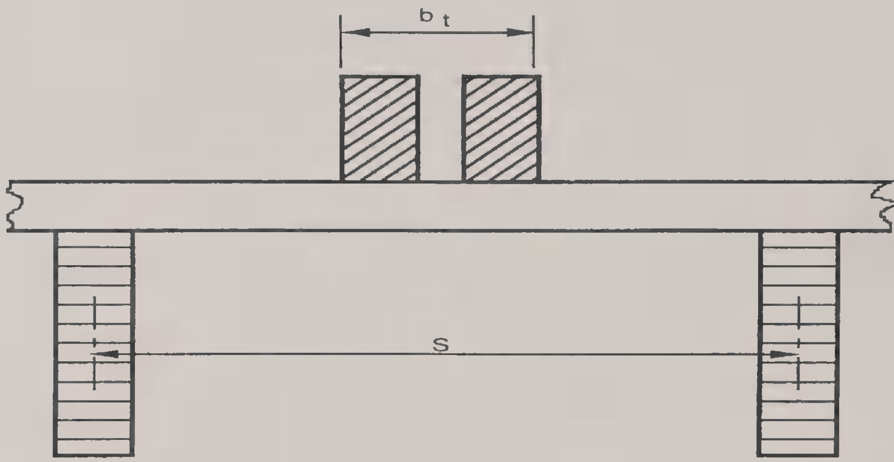
Assuming 1/4" joint between panels, then

L = (40.625 x 2) + 0.25 + (7 x 41.75) + (7 x 0.25) = 375.5 in.
L = 375.5in. = 31.29' ≈ 31.3'

7 - 41 3/4" Deck Panels
2 - 40 5/8" Deck Panels
1/4" Space Between Panels

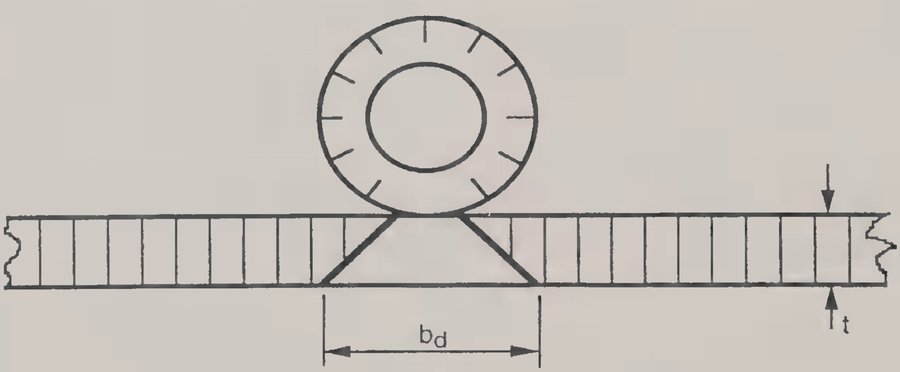
3.13.2 Determine Effective Deck Section

In the direction of deck span, b_t. AASHTO 3.25.1.1



$$b_t = \sqrt{0.025\ P} = \sqrt{0.025 \bullet 15{,}000} = 19.36 \text{ in.}$$

In the direction normal to deck span, b_d,



$b_d = 15 + t = 15 + 4 = 19$ in. << Panel Width

Section properties:

$$A = b_d \times t = 19 \times 4 = 76 \text{ in.}^2$$

$$S_y = \frac{b_d \times t^2}{6} = \frac{19 \times 4^2}{6} = 50.7 \text{ in.}^3$$

$$I_y = \frac{b_d \times t^3}{12} = \frac{19 \times 4^3}{12} = 101.3 \text{ in.}^4$$

3.13.3 Calculate dead load and live load moments, shears, and deflections

Dead load:

$$W_{DL} = 90.83 \text{ psf x } \frac{1}{144} \text{ x } 19 \cong 12.0 \text{ lb./in.}$$

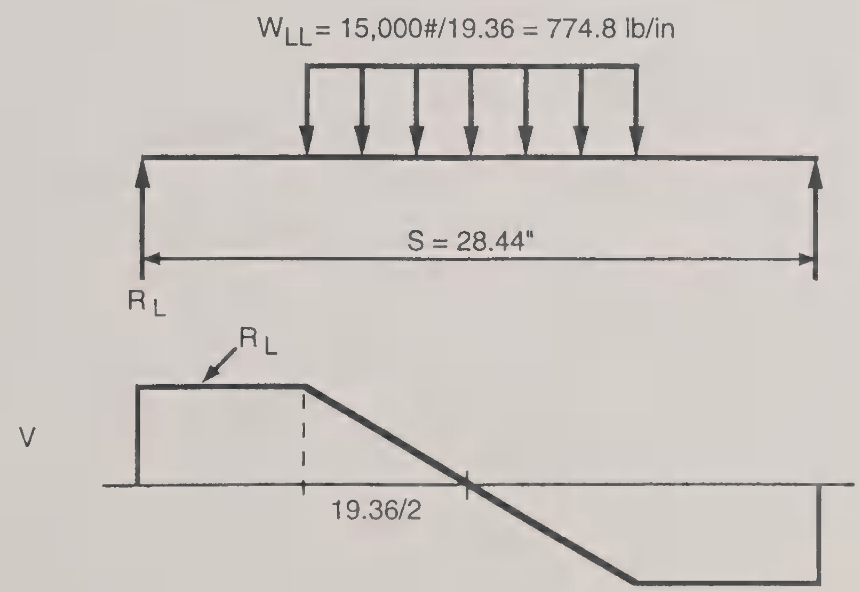
$$M_{DL} = \frac{W_{DL} \bullet S^2}{8} = \frac{12.0 \bullet 28.44^2}{8}$$

$$M_{DL} = 1212 \text{ lb. - in.}$$

$$V_{DL} = W_{DL}\left(\frac{S}{2} - t\right) = 12.0\left(\frac{28.44}{2} - 4\right)$$

$$V_{DL} = 122.6 \text{ lb.}$$

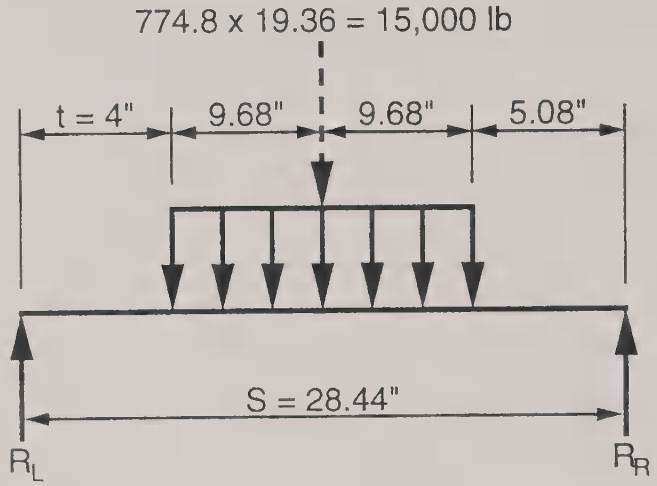
Live Load: HS-25 or IML Loads



$$M_{LL} = R_L\left(\frac{S - 19.36}{2}\right) + \frac{1}{2} R_L \frac{19.36}{2}$$

$$M_{LL} = 7500\left(\frac{28.44 - 19.36}{2}\right) + \frac{1}{4}(7500)(19.36)$$

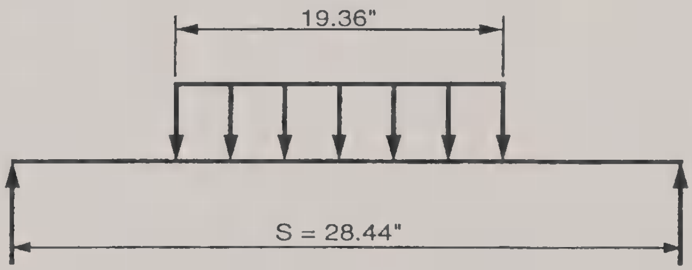
$$M_{LL} = 70{,}350 \text{ in. - lb.}$$



$$V_{LL} = R_L = \frac{15{,}000\ (9.68\ +\ 5.08)}{28.44}$$

V_{LL} = 7785 lb.

Δ_{LL} in the same position as for moment



$$\Delta_{LL} = \frac{1.80}{E_y{}'I_y}[138.8\ S^3 - 20,780\ S\ +\ 90,000]$$

Ritter, USDA Forest Service

$E_y{}' = 1.6 \times 10^6 \cdot 0.833 = 1.332 \times 10^6$

$I_y = 101.3$ in.^{4}

$\Delta_{LL} = 0.036$ (For 12,000 lb. load)

$\Delta_{LL} = \frac{15}{12}$ (0.036) = 0.045 in. (For 15,000 lb. load)

3.13.4 Check actual stress against allowable design values

Allowable stresses:

F_{by} = 1800 psi	C_m = 0.800	BLC 560
F_{vy} = 215 psi	C_m = 0.875	NDS, 1991
E_y = 1.6×10^6 psi	C_m = 0.833	PSU Research

Calculated stresses:

Total Moment = $M_{DL} + M_{LL} = 1{,}212 + 70{,}350$ Continuous over more than two spans, AASHTO

$M_{TOT} = 71{,}562$ in.-lb.

$$f_{by} = \frac{0.80\ M_{TOT}}{S_y} = \frac{0.80 \bullet 71{,}562}{50.7}$$

$f_{by} = 1129$ psi

$F_{by}' = 1800 \times 0.8 = 1440 > 1129$

Total Shear = $V_{DL} + V_{LL} = 122.6 + 7785$

$V_{TOT} = 7{,}908$ lb.

$$f_{vy} = \frac{3 \bullet 7908}{2 \cdot 76} = 156 \text{ psi}$$

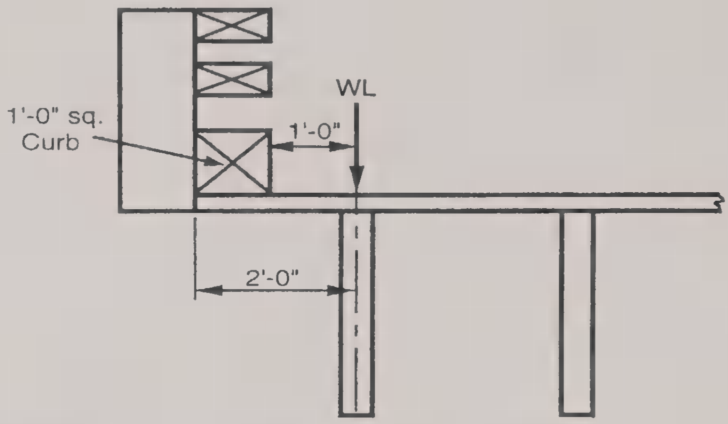
$F_{vy}' = 215 \bullet 0.875 = 188 > 156$

Deflection:

$\Delta_{LL} = 0.045" < 0.10$ in.

3.13.5 Check Overhang

To check the overhang, the wheel load is placed a distance equal to 1'-0" from the face of the curb. The wheel load is directly over the outside girder for this example, therefore, there will be no live load added to the overhang.



3.13.6 Size Dowels for this Deck Thickness

AASHTO 3.25.1.3

$$n = \frac{1000}{\sigma_{PL}}\left\{\frac{R_y}{R_D} + \frac{M_y}{M_D}\right\} = \text{Number of dowels}$$

where:

$\sigma_{PL} =$ 1010 for N. Red Oak

Wood Handbook, P. 4-10

$$R_y = \frac{6\ PS}{1000} \text{ for } S \leq 50 = \frac{6 \bullet 15{,}000 \bullet 28.44}{1000} = 2560$$

$$M_y = \frac{PS\,(S - 10)}{1600} = \frac{15{,}000 \bullet 28.44\,(28.44 - 10)}{1600} = 4917$$

R_D and $M_D =$ shear and moment capacity of dowel, respectively.

Assume: 3/4" ϕ dowel

$$n = \frac{1000}{1010}\left\{\frac{2560}{1020} + \frac{4917}{1960}\right\} = 4.96 \approx 5 \text{ dowels/panel between stringers}$$

Since this requires dowels @ 6 in. o.c., try larger dowels.

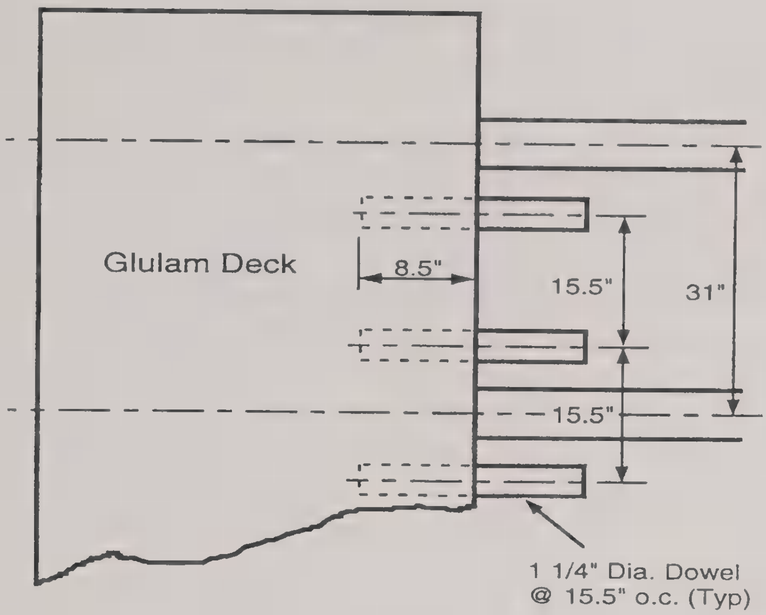
3.13.7 Modify Dowel Size and Spacing

Try 1-1/4" ϕ dowels

$R_D = 2100$ $\qquad$ $M_D = 5950$

$$n = \frac{1000}{1010}\left\{\frac{2560}{2100} + \frac{4917}{5950}\right\} = 2.03 \text{ dowels}$$

Use two 1-1/4” dowels equally spaced between the center-line of the girders.



3.14 Design Summary

Northern Red Oak Glulam Superstructure

30'-0" between bearing centerlines

Creosote treated

11-5 1/8" x 39" stringers @ 2'-7" o.c.

7 - 4" x 41 3/4" deck panels to be placed in the middle

2 - 4" x 40 5/8" deck panels to be placed on the ends

1/4" joint between panels

2 - 3 1/8" x 39" x 30' backwall diaphragms

10 - 3 1/8" x 34 1/2" x 25 7/8" center line diaphragms

3.15 Alternate Stringer Sizes

The stringers seem to be very deep, but may be smaller for alternative load criteria, wet use criteria and beam widths.

Modification	Beam Size	% Beam Depth Decrease
HS20 Load, C_m = 0.80	5 1/8" x 33"	15.4%
HS20 Load, C_m = 1.00	5 1/8" x 27"	30.8%
Alternate Military, C_m = 1.00	5 1/8" x 30"	23.1%
HS25 Load, C_m = 1.00	5 1/8" x 33"	15.4%
HS25 Load, C_m = 1.00 b = 6 3/4	6 3/4" x 28 1/2"	3.8%

NORTHERN RED OAK DEMONSTRATION BRIDGE

4.1 Overview

A demonstration bridge project has been underway in Pennsylvania for the past several years. The goals of this effort are to design, construct, and monitor hardwood timber highway bridges throughout the state, thus demonstrating the suitability of hardwoods for structural components in highway bridges. To date, several hardwood transverse stressed, longitudinal decks, both unreinforced and steel-plate reinforced, demonstration bridges have been completed. At least three of the proposed demonstration bridges are to be hardwood glulam bridges, one each of northern red oak, red maple, and yellow poplar. The objective of the remainder of this section is to summarize the design of and field performance of a northern red oak hardwood glulam demonstration highway bridge which was completed and opened for traffic in November 1991. Design details are further described in Manbeck, et al. (1991).

4.2 The Project Team

The project was a cooperative effort of several organizations under the leadership of a Penn State University Research Team from the Departments of Agricultural and Biological Engineering and the Wood Products Program of the School of Forest Resources. The Penn State Research Team was responsible for all quality control matters and specifications related to wood procurement, processing, grading, and fabrication. Gwin Dobson and Forman, Inc. of State College, Pennsylvania, designed the substructure and superstructure and supervised construction; Unadilla Laminated Products, Inc. of Sidney, New York, fabricated the glued laminated structural members and provided fastener hardware; Koppers, Inc. of Muncy, Pennsylvania, treated the glued laminated members, and Kamtro Construction of Osceola Mills, Pennsylvania, constructed the bridge. The bridge owner is Ferguson Township in Centre County, Pennsylvania.

4.3 Design Requirements and Procedures

A northern red oak glued laminated girder and glued laminated deck was designed to replace a 44-year-old reinforced concrete tee beam bridge with a 107 kN (12 ton) rating on Township Road T-330 in Ferguson Township in Centre County, Pennsylvania. The bridge superstructure was erected onto the existing stone abutments. The bridge skew, at 45 degrees, was severe.

The design requirements for the bridge were:

- Loads - HS25 or ML80 live load
- Deflections - Live load deflection less than span/500
- Materials - All superstructure, railings, and parapets to be glued laminated northern red oak
- Clear span between centerline of abutments - 10.69 m (35 ft. 0 1/2 in.)
- Overall deck width - 8.54 m (28 ft.)

All structural components were designed in accordance with the 1986 ed. of the *National Design Specification for Wood Construction* (NFPA, 1986), the 1988 ed. of the Supplement to the *National Design Specification* (NFPA, 1988), the *AASHTO Standard Specifications for Highway Bridges*, (AASHTO1989), and PennDOT Design Manual Part 4 (PennDOT 1990). All the girders were specified as Combination A lay-ups (Fig. 4.1) with the following unadjusted structural properties: F_{bx} = 15.4 MPa (2240 psi); F_v = 1.5 MPa (230 psi); E = 11.0 GPa (1.6 x 10^6 psi). The girders were braced laterally by endwall diaphragms, midspan diaphragms and by the glulam deck which was fastened to the girders every 0.30 m (12 in.) on center. The glued laminated deck panels were specified as Combination A northern red oak with F_b = 15.4 MPa (2240 psi), F_v = 1.5 psi (230 psi), and E = 11.0 GPa (1.6 x 10^6 psi).

4.4 Bridge Design

The bridge superstructure has nine 203 mm by 743 mm (8 in. by 29-1/4 in.) girders spaced 965 mm (38 in.) on center (Fig. 4.2). All girders were fabricated with 38 mm (1.5 in.) laminations. The 152 mm (6 in.) thick deck consists of 914mm (36 in.) and 1220 mm (48 in.) wide by 8.54 m (28 ft.) long panels. All panels are spaced approximately 13 mm (1/2 in.) apart to accommodate anticipated in-service moisture expansion because the panels were fabricated at 12±2% moisture content and are expected to equilibrate over the stream at 16 to 19% moisture content. The 152 mm (6 in.) deck was designed as a non-interconnected deck (AASHTO, 1989, Ritter, 1990). However, one-half of the bridge was constructed with 32 mm (1 1/4 in.) diameter dowels to observe performance differences, if any, between the asphalt paving over the <u>interconnected</u> panels and the <u>non-interconnected</u> panels. The endwall diaphragms were 152 mm (6 in.) wide by 743 mm (29 1/4 in.) deep and extended the full 12.08 m (39.6 ft.) skew length. Midspan diaphragms, 150 by 743 mm (3 in. by 29 1/4 in.), were installed perpendicular to the span between each pair of girders for lateral stability. The girders were attached to the abutment with 19 mm (3/4 in.) anchor bolts (all bridge hardware was galvanized). The bearing design allowed vertical adjustment for proper leveling of the top surfaces of the nine beams. The deck panels were fastened to the girders with 19 mm by 229 mm (3/4 in. x 9 in.) galvanized lag bolts. The heads were recessed into the deck. The diaphragms were connected to the girders with three 19 mm by 229 mm (3/4 x 9 in.) galvanized lag bolts at each girder. (This detail has been changed in the BLC-560 Standard Plans (PennDOT 1994) to 2-19 mm (3/4 in) diameter threaded rods which extend through the diaphragm and two adjacent beams.)

Oakum was installed between deck panels to prevent asphalt paving from filling the space. Before paving, a waterproof geotextile membrane was installed over the deck. (This detail is not included in the new BLC-560 Standard Plans (PennDOT 1993)).

The railings and parapets design consists of 254 mm by 305 mm (10 in. x 12 in.) glued laminated posts spaced 1.83 m (6 ft.) on center, two 152 mm by 203 mm (6 in. x 8 in.) glued laminated rails, and 254 mm x 305 mm (10 in. x 12 in.) glued laminated curbs. The rail system is fastened with galvanized bolts and drift pins and is similar in design to that tested by Ritter, et al. (1991).

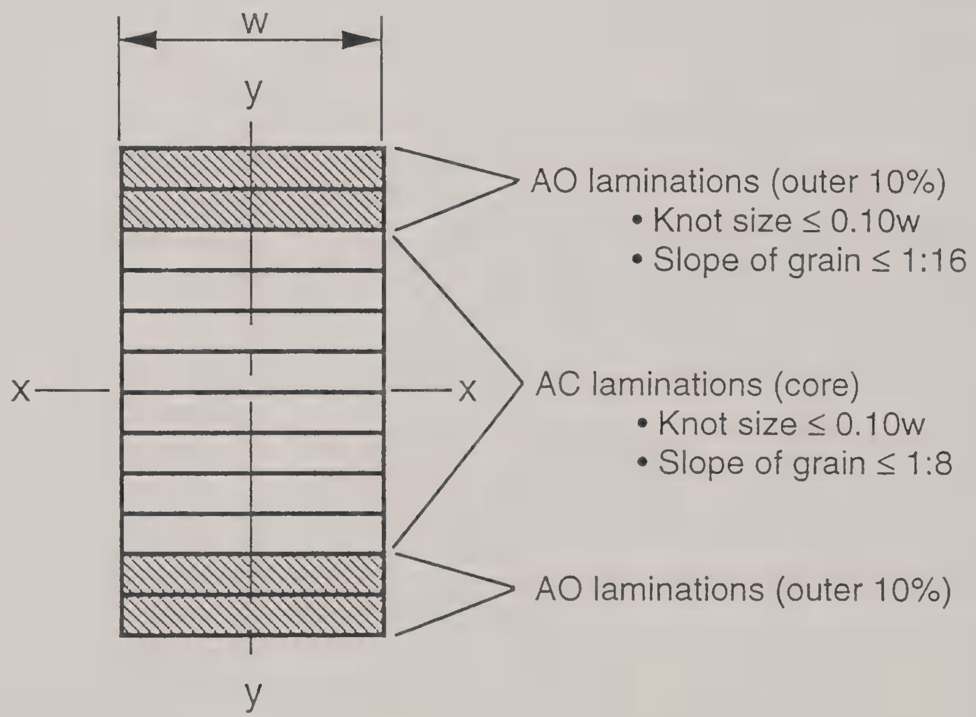


Figure 4.1. Glulam beam layup (Combination A) for Northern Red Oak Girders.

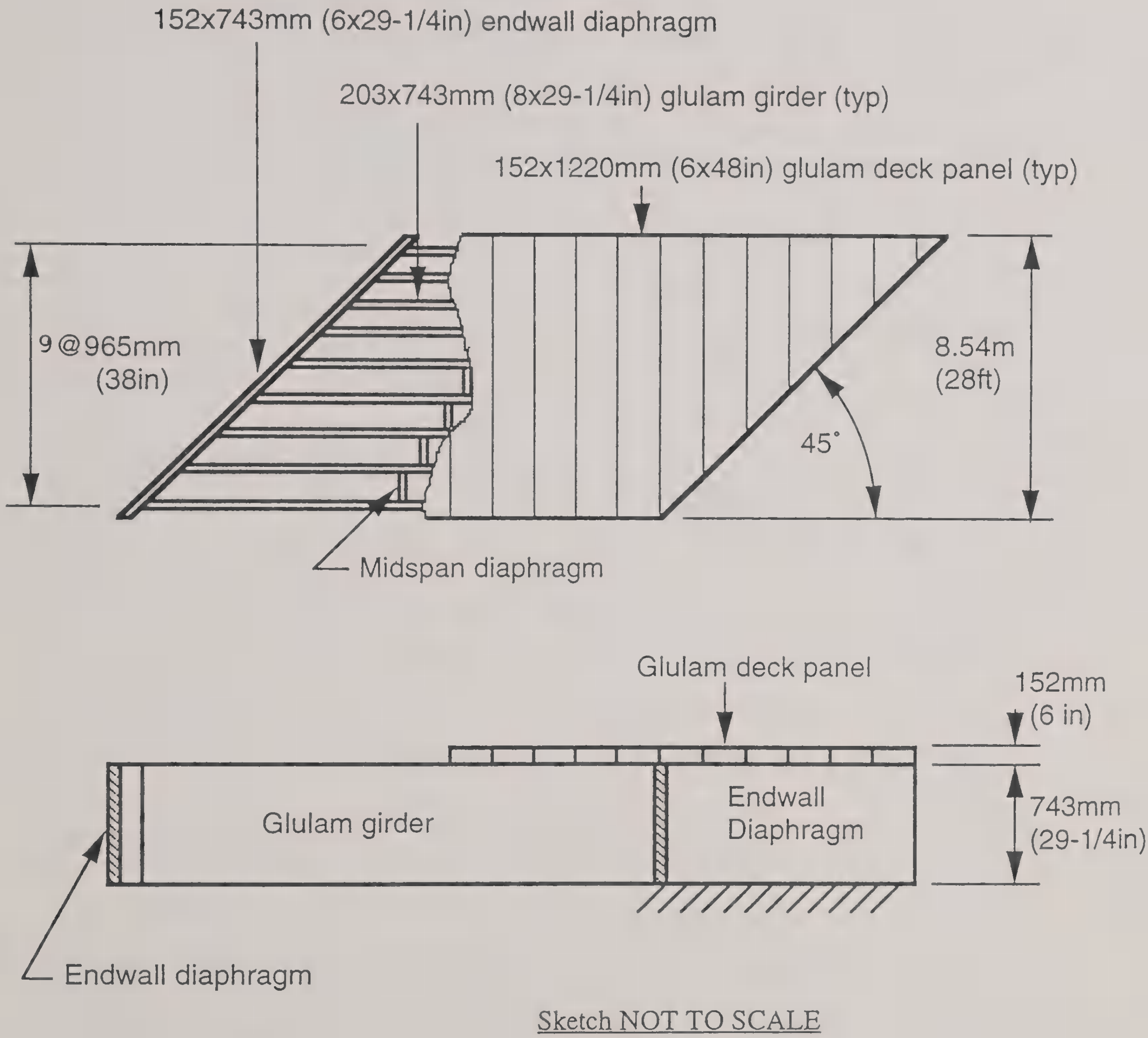


Figure 4.2. Sketch of superstructure layout for the 10.69 m (35 ft-1/2 in) clear span northern red oak glulam demonstration bridge.

4.5 Fabrication and Construction

Most hardwood lumber is not structurally graded nor is hardwood generally available in standard dimension sizes as are softwoods. The current lack of such materials is a major challenge to the use of hardwoods in glulam applications. However, traditional hardwood manufacturers have shown interest and are exploring the production option for structurally graded dimension lumber.

Members of the Penn State Research Team procured the northern red oak logs, arranged for primary processing and drying of the lumber, and then sorted and graded the lumber in accordance with AITC's hardwood laminating specifications, AITC-119 (AITC, 1986). The team also supervised and oversaw the fabrication of the girders, deck panels, and railing materials at Unadilla Laminated Products, Inc. in Sidney, New York. As part of the fabrication process, Unadilla planed and cut all of the glulam members to the required finished dimensions to minimize field cutting after preservative treatment. The following holes were predrilled at Unadilla prior to preservative treatment: the deck holes for the deck to girder fastening; the girder holes for the girder to midspan diaphragms; the backwall holes for the back wall to girder connections, the holes in the railings, posts, and curbs. All other holes were field drilled and field treated with creosote. Field drilled holes were swabbed thoroughly with creosote and subsequently filled with asphaltic roof cement prior to inserting the fastener. All bridge connection hardware was fabricated and supplied by Unadilla, Inc. The team then shipped the laminated materials to Koppers, Inc., Muncy, Pennsylvania, where they were treated with creosote to a retention level of approximately 192.2 kg/m^3 (12 pcf) with a minimum depth of penetration of approximately 6 mm (0.25). Finally, the team shipped the glued laminated members and hardware to the bridge site.

The notice to proceed for construction was given on September 6, 1991. Demolition of the existing concrete bridge began on September 9, 1991. The bridge bearing seat caps were reconditioned by September 23. Much of the rip rap and gabion work was completed by September 30. The girders were set and diaphragms attached by October 4, 1991. Railings and parapets and all paving were completed by October 28. On October 30, the bridge was load tested with two 75,000 lb. triaxle trucks. On November 1, eight weeks after the notice to proceed, the road was reopened to traffic.

Special care was necessary in the construction phase to assure proper mating of the deck to the girder. All deck panels had to be properly mated to the girder prior to insertion of and tightening of the lag bolts. This can be accomplished by careful sequencing of the fastener application and by preloading the deck panel with construction equipment prior to fastening and properly torquing the lag bolts.

Dimensional stability was also a concern in the fabrication/construction phase. Laminating procedures require lumber to be at a moisture content of less than 14%. In service treated lumber in the deck panels is expected to equilibrate at 16 to 19% moisture content. Thus, adequate spacing, the magnitude of which is somewhat species dependent, must be specified between deck panels. Noting that some dimensional expansion is observed after creosote treatment, panel widths at the site should be measured prior to installation for necessary panel spacing adjustments in the field.

4.6 Cost

The total cost of the bridge, including all materials, development, and monitoring costs, was approximately \$250,000. The design, construction, and engineering costs, which totaled \$140,000, were higher than normal for a "spec" bridge. Laminating costs were \$40,000. Since this bridge was a demonstration/experimental bridge, it was the first of its kind. Consequently, more uncertainties with respect to design, fabrication, and construction procedures required more time and effort. Once the standard designs and specifications are available in 1994, these costs should decline significantly, thus making timber bridges more cost competitive. In addition, these costs include proof loading and initial monitoring, which would not be required of a "spec" bridge.

4.7 Performance Test Results

The predicted live load deflection, assuming no composite behavior between the deck and girder, girder E-value of 11.0 GPa (1.6 x 10^6 psi), and an HS25 or ML80 load, was 22 mm (0.85 in.). A load test with two 33.4 kN (75,000 lb.) triaxial trucks, located so as to produce maximum deflection, resulted in an actual maximum deflection of 14 mm (0.55 in.). Lower actual vs. predicted deflections is probably due to: 1) The conservative design value of E (Shaffer, et al. (1991) reported E-values of 13.1 GPa (1.90 x 10^6 psi) for northern red oak beams) used in the calculations; 2) Neglect of composite action between the deck and girders, and 3) The average E-value, determined by static loading of each board, used in the bridge girders being 15.5 GPa (2.2 x 10^6 psi.). Predicted live load deflection using an E-value of 13.1 GPa (1.9 x 10^6 psi) and 15.5 GPa (2.2 x 10^6 psi) equals 18 mm (0.72 in.) and 16 mm (0.62 in), respectively. Twenty-one month's data indicate that creep deflections are negligible (less than 1 mm).

4.8 Hardwood Glulam Bridge Standards

In 1994, the Pennsylvania Department of Transportation will publish the BLC-560 Series *Standards for Hardwood Glulam Timber Bridge Design* (PennDOT, 1994) for clear spans of 5.49 to 27.4 m (18 to 90 ft.) The standards include designs and specifications for the design, fabrication, treatment, handling, and erection of components and assemblies for hardwood glulam bridge superstructures and wood, concrete, or steel substructures. The standards are flexible and include provisions for hardwood or non-hardwood substructures used in conjunction with hardwood superstructures. The standards also include selection tables for the glulam girders. Notable features of the BLC-560 series are a worked design example and design worksheets.

4.9 Summary

A demonstration highway bridge using northern red oak glulam members for the superstructure, including parapets and railings, has been successfully designed, fabricated, erected, and load tested. Standard designs (BLC-560 series) are near completion for northern red oak, red maple, and yellow poplar glulam bridges with clear spans ranging from 5.49 to 27.4 m (18 to 90 ft.). The standards will be available from the Pennsylvania Department of Transportation.

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NOTES

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